LANDAU
ASSOCIATES,
INC.
Geoenvironmental Engineering and Technologies

7.2.1

November 14, 1991

Mr. Neil Thompson
U. S. Environmental Protection Agency
Park Place Building
1200 Sixth Avenue
Seattle, WA 98101

RE: **RESPONSES TO EPA/ECOLOGY COMMENTS TO COLBERT LANDFILL REMEDIAL DESIGN/REMEDIAL ACTION PHASE I ENGINEERING REPORT**

Dear Mr. Thompson:

Contained herein are responses to U.S. Environmental Protection Agency (EPA), Washington State Department of Ecology (Ecology), and Ecology & Environmental (E&E; EPA's oversight contractor) comments on the (Draft) Colbert Landfill Remedial Design/Remedial Action Phase I Engineering Report (ER). EPA did not provide internal written comments on the ER, but did provide verbal comments during the EPA/Ecology/Spokane County September 26, 1991 meeting in Seattle, Washington. Ecology's written comments are dated October 8, 1991, and E&E's comments are dated September 9, 1991.

Written responses to review comments have been prepared by Landau Associates, Inc. (Landau Associates), Spokane County's engineering consultant for the Colbert Landfill Remedial Design/Remedial Action project. Comments and responses are presented below, and are formatted in the same order as presented in the Ecology and E&E comment letters. Responses to verbal comments from the September 26, 1991 meeting are presented in the approximate order of discussion.

At the request of Ecology, the ER will not be revised until EPA and Ecology have had the opportunity to review the proposed revisions. To facilitate this review, text for ER sections that were significantly revised in response to comments are included with this comment letter as attachments (revisions are underlined). Revised Section 5.0 is presented in Attachment A, revised Sections 4.2 through 4.4 are presented in Attachment B, and revised Appendix E is presented in Attachment C. The substance of the proposed revision is contained within the response to comment for revisions not provided in the attachments to this letter. The ER will be finalized subsequent to EPA and Ecology review of this letter and concurrence with the proposed revisions.

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RESPONSE TO ECOLOGY'S OCTOBER 8, 1991 COMMENTS

1. Comment: Statements on page 4-35 imply that Ecology and EPA must approve the Phase I report before Phase II design begins. However, as written, the report contains interpretative conclusions and opinions not necessary for Phase II, but which may be influential regarding other compliance issues and legal matters. Also, we are unable to substantiate some of the conclusions, and disagree with some of the opinions. Consequently, it will be very difficult for Ecology to approve the report unless significant revisions are made.

   Ecology could concur with the design parameters, i.e. transmissivity, storage coefficient, etc. However, we find no summary of design parameters. We are willing to provide reassurances for Phase I, and suggest discussing this topic at a mutually agreeable time.

   Response: The statement on page 4-35 was modified to indicate that Phase I design will proceed following EPA and Ecology review of the ER and approval to proceed with Phase II design. A statement in Section 5.5 (revised Section 5.6.1) was modified using similar wording.

   Although we understand that EPA and Ecology will not approve the ER, it is important that concurrence be obtained for certain design parameters and conditions. Section 5.5 (revised Section 5.6.1) was expanded to present the conclusions that need EPA and Ecology concurrence (see Section 5.6.1 in Attachment A).

2. Comment: Much of the Executive Summary provides rationale for why the South, West, and East extraction systems will be different from the conceptual model provided in the ROD. However, the differences themselves are not described or identified. As these differences represent a change in thinking since the ROD was signed, we are interested in the magnitude of the differences. Please describe the differences quantitatively, to the extent possible, and display the differences conceptually on Figure ER 1.2.

   Response: A revised version of Figure ER-1.2 (Figure ER-5.1) was added to Section 5 (and was added to the Executive Summary) to indicate the anticipated differences between the ROD and Phase II conceptual designs. It should be recognized that the Phase II design shown on Figure ER-5.1 is conceptual and that as noted in Section 5.2, will probably require some level of modification during Phase II design (see Figure ER-5.1 in Attachment A).

3. Comment: The last paragraph of the executive summary implies something is abnormal with Phase II. The statement is made that the design of Phase II requires early estimates of loading estimates and constituent concentrations, but it is later stated that these estimates are normally developed in latter stages of design. Please elaborate on the timing of the loading estimates. In particular, does the timing pose any technical concern for an efficient cleanup of the site?

   Response: The last paragraph of the Executive Summary discusses a timing issue for Phase II design presented in Section 5.5 (revised Section 5.6.1) of the main body of the ER. The issue is whether the interception system design and the treatment system design should be submitted concurrently (as presently scheduled), when much of the interception system design must be completed prior to design of the treatment system.

This is primarily a concern because treatment for scale control will be required for the Phase II treatment facility and will significantly increase the level of effort for Phase II treatment system design. This additional level of effort was not anticipated during development of the Schedule For Submittal Of Deliverables. However, during the September 26, 1991 meeting EPA identified this as a scheduling issue that needs to be addressed outside the context of the ER. Consequently, the reference to a potential change in Phase II design submittals was deleted from Section 5.5 (revised Section 5.6) and from the Executive Summary. If it becomes necessary to revise the Schedule For Submittal Of Deliverables for Phase II activities, a request for such a change will be submitted to EPA and Ecology for review and approval.

4. Comment: Phase I has expanded the understanding of hydrogeology defined in the Remedial Investigation (RI), and in some cases has provided new information sufficient to alter the conceptional model of the extraction system provided in the ROD. Section 4.2 states that these differences may even impact Phase II design. Throughout the report, references are made to an individual change in RI hydrogeology, but each reference provides only a piece of the picture. A comprehensive summary of how the hydrogeology in this report differs from the RI hydrogeology should be provided in a separate section to demonstrate why the Remedial Investigation is no longer the definitive reference for hydrogeology.

   Response: A new subsection was added to the conclusions and recommendations (Subsection 5.1) to describe the substantive differences between the hydrogeologic characterization presented in the RI and that resulting from the Phase I investigation (see Subsection 5.1 in Attachment A).

5. Comment: A section discussing the environmental permitting requirements for the project should be made in the Phase I report. If substantive requirements rather than administrative requirements are to be met they should be identified. In particular, estimate for loading to the Little Spokane River from treatment discharge should be provided. Estimates for air emissions should also be provided.

   Response: A new subsection has been added to the conclusions and recommendations (Subsection 5.6.4) to address Phase II permitting. Subsection 5.6.4 addresses anticipated permitting requirements that require substantive compliance. Analyses and submittals required to demonstrate substantive compliance (including estimated river and air loadings) will be developed during Phase II design and are not addressed within the ER (see Subsection 5.6.4 in Attachment A).

6. Comment: What is the fate of the infiltration system used for testing the South Systems Pilot Facility? If the system remains as is, will it be shown on the deed to the property?

   Response: The South System infiltration system was abandoned by capping the influent line below the ground surface. The system is presently intact and could be used (at the property owner's discretion) for a septic system leach line or other purposes, provided it meets State and local requirements for such use. To the best of our knowledge, there is no intent for the infiltration system to be shown on the deed to the property.

7. Comment: Is the infiltration system to be used again? If not, then please make a statement to that effect. Use of the infiltration system for long term operation would most likely require a state permit.

   Response: There is no intent to use the South System infiltration system during Phase II activities. Section 5.0 was modified to include a statement to that effect (see Section 5.5 in Attachment A).

8. Comment: The impact of the estimated 2000 gallons per minute discharge to the Little Spokane River should be discussed.

   Response: The discharge of 2,000 gallons per minute to the Little Spokane River should not have a significant impact on river flow. A statement to that effect was incorporated into Section 5.0 (see Section 5.6.3 in Attachment A).

9. Comment: Please substantiate the conclusion made in Section 4.3.1 that compounds detected in the lower aquifers other than the Constituents of Concern were present in low concentrations and only in a limited number of wells. How is "low" defined? What is a "limited" number of wells?

   Response: A table was added to Section 4 (Table ER-4.4) summarizing the detected compounds, number of detections, and maximum concentration detected. The text was modified, and now refers the reader to the complete summary of the analytical results in Appendix F from Section 4.3.1 (see Table ER-4.4 in Attachment B).

10. Comment: A single round of sampling is normally not sufficient to eliminate the search for detected contaminants in a drinking water aquifer at a Superfund site. Please explain in Section 4.3.1 the rationale for eliminating contaminants, other then the contaminants of concern, from future sampling rounds when those contaminants were detected in the first round. What contaminants were detected? What were their concentrations? Why were they eliminated?

    Response: The Constituents of Concern for the Colbert Landfill (Landfill) project were previously defined during the RI. Additionally, the Consent Decree Scope of Work specifies that volatile organic analyses need only include the six Constituents of Concern. The Field Sampling Plan [contained within the EPA and Ecology approved Quality Assurance Project Plan (QAPjP)] specifies that volatile organic analyses would be reduced to the six Constituents of Concern after the first round of groundwater sampling, except at locations where other constituents were detected at significant concentrations (QAPjP, page FS-3-2). Section 4.3.1 was modified to include a reference to the previously agreed upon criteria presented in the QAPjP. Additional modification in response to this comment is not proposed (see Section 4.3.1 in Attachment B).

    The summary table presented in response to the previous comment should clarify the additional questions regarding the contaminants detected and their concentrations.

11. Comment: One reason given in section 3.3 for the lack of accuracy and consistency of the geophysical survey is higher conductivity of groundwater associated with landfill leachate (page 3-4). This reason implies a leachate problem associated with the landfill. We find no support in the report for this reason. Either provide support, strike the reason, or outline steps to investigate the problem.

Response: The reference was deleted from the ER.

12. Comment: Regarding the discussion of landfill leachate in section 4.3.4 the statement is made that monitoring wells CD-30A and CD-21C1 are located in areas where landfill leachate would be anticipated and that Chloride, hardness, TDS, TOX, calcium and conductivity in these wells are slightly elevated while the pH is somewhat lower than normal. These statements of judgement (i.e. "slightly elevated" and somewhat lower than normal") require the identification of the specific data on which they are based.

Response: The text was modified, and the reader is now referred to the analytical results presented in Table ER-4.5 within the discussion in Section 4.3.4 regarding the relative concentration of landfill leachate parameters (see Section 4.3.4 in Attachment B).

13. Comment: Please strike the word "apparently" from the first sentence of the last paragraph on page 3-8. Use of apparently in describing whether or not the criteria were exceeded contradicts the last paragraph and the proceeding paragraph which describe how the criteria were exceeded.

Response: The word "apparently" was stricken from the last paragraph on page 3-8.

14. Comment: Figure ER 1-3 is labeled as a Pre-Project Geologic Schematic. "Pre-Project" is a nondescript term. As the geology in the figure comes from the Remedial Investigation, we ask the figure be labeled "Remedial Investigation Geologic Schematic" and that a date for the interpretation be provided.

Response: The figure was modified to reflect that it is based on data collected during the 1987 RI.

15. Comment: In Section 4.2.1 the statement is made that aquifer parameters are overestimated and that this over estimation is appropriate for Phase II. Please identify in section 4.2.1 or in section 4.2.3.2 which aquifer parameter are over estimated. We do not agree with the blanket statement that over estimated parameters are appropriate for Phase II design. For example, a low value of transmissivity seems more appropriate then a high value when determining the spacing for extraction wells. Appendix E does not give a summary of which parameters are overestimated.

Response: Section 4.2.1 does not state that aquifer parameters are overestimated. What is stated is that analysis methods that tend to overestimate transmissivity were included in the analyses and, as a result, the average values represent a reasonable upper bound. It is primarily transmissivity that tends to be overestimated using these straight line techniques, although storativity may be overestimated in some cases. Section 4.2.1 was modified to state this more clearly. Ecology is correct in pointing out that overestimation (or using a reasonable upper bound) is not appropriate for all aspects of the Phase II interception system design. This is particularly true for the Upper Sand/Gravel Aquifer, where a limited saturated thickness is available and potential overestimation of aquifer properties cannot be compensated for by providing excess available head. Section 4.2.1 was modified to clarify the design considerations associated with developing aquifer parameters. Additionally, the text was modified and now provides lower bound transmissivity values for Upper and Lower Sand/Gravel Aquifers in Sections 4.2.3.2 and 4.2.4.2, respectively, and Appendix E of the ER (see Sections 4.2.3.2 and 4.2.4.2 in Attachment A and Appendix E in Attachment C).

16. Comment: In regard to the statement made at the end of the fourth paragraph on page 4-12, a high transmissivity or the dampening of percolation could also explain the lack of seasonal water level fluctuation. Why was the lack of fluctuation attributed to a significant recharge source? What other possible causes were rejected?

Response: We concur that high transmissivity or dampening of percolation are potential causes for limited seasonal fluctuation. However, the annual fluctuation for the Upper Sand/Gravel Aquifer appears to be only about 0.2 ft, which is extremely small for an aquifer of apparently limited areal extent, given the seasonal nature of aquifer recharge in the Spokane area. Elevation controlled discharge boundaries could also reduce the impact of seasonal fluctuation, although this also does not appear to be an adequate explanation for such small seasonal fluctuations. It is possible that water level data collected during Phase II operations will clarify the boundary conditions, or other influences, that cause the limited response of the Upper Sand/Gravel Aquifer to seasonal recharge. Modification to the ER text is not proposed in response to this comment.

17. Comment: In section 4.2.3.2 it is stated that step drawdown tests were not performed due to treatment limitations. However, step drawdown tests have been necessary to determine the efficiency of the extraction wells at many other sites, and are a routine step in long term, high yield wells. If step drawdown tests are not to be performed please make a statement to that effect and justify why the tests are not going to be performed.

Response: Step drawdown tests could be performed during Phase II when treatment system operation does not depend on a single well. However, because of the continuous and long-term nature of pumping for Phase II, Phase II design entrance velocities will be established that are significantly below those typically employed for high-yield wells. As a result, well efficiency will not be a significant concern, other than changes in efficiency over time. Consequently, it is not anticipated that step drawdown testing will be required for Phase II. Sections 4.2.3.2 and 4.2.4.2 were modified to reflect these considerations (see Sections 4.2.3.2 and 4.2.4.2 in Attachment B).

18. Comment: In regard to the conclusion drawn in the last paragraph of section 4.2.4.1 that the Little Spokane River appears to be the primary source of discharge for the lower sand and gravel aquifer, we do not find the single gradient measurement convincing. Also, we do not follow the logic that the river is primary source of discharge based on the

elevation difference between the water level in well CD-40 and the river stage at Dartford. (Where is Dartford?) Please clarify the evidence for the conclusion.

Response: The statement regarding the Little Spokane River appearing to be the primary discharge location for the Lower Sand/Gravel Aquifer is primarily a reiteration of the conclusion drawn in the RI (RI Section 5.2.1.3). The purpose of comparing the river stage at Dartford (Dartford is about 4 miles downstream from the Phase I river outfall) is to demonstrate that an upward gradient exists from the groundwater to the river. Although a single gradient measurement may not be convincing, it further supports the conclusion originally drawn in the RI and apparently accepted at that time by EPA and Ecology. Because the extent to which the Little Spokane River is the primary discharge location for the Lower Sand/Gravel Aquifer does not significantly impact Phase II design, revision of the ER is not proposed in response to this comment.

19. Comment: In regard to calculation of well efficiency in section 4.2.3.2 by use of distance drawdown, were the effects of partial penetration taken into account? Also, we require the calculations for all well efficiency estimations in order to validate the efficiency.

Response: The effects of partial penetration were not taken into account for well efficiency estimates. Well efficiency for Extraction Wells CP-S1, CP-W1, and CP-E1 were reevaluated using drawdown data corrected for partial penetration using an analysis method presented in Driscoll (1986, pages 575-579). The revised analyses indicate well efficiencies of 64 percent, 85 percent, and 78 percent for Wells CP-S1, CP-W1, and CP-E1, respectively. Well efficiency was not estimated for CP-E2 because it was completed "open hole" in basalt. The main body of the text and Appendix E of the ER were revised to reflect these analyses (see Sections 4.2.3.2 and 4.2.4.2 in Attachment A; see Section 2.1 (14) in Attachment C for a description of the well efficiency evaluation method, and the well efficiency calculations are now provided on the appropriate figures provided in Attachment C).

20. Comment: In regard to the calculated well efficiencies of 40 to 50 percent, the statement is made on page 4-16 that actual well efficiencies are expected to be higher. How much higher are efficiencies expected to be? What value of well efficiency will be used for Phase II design purposes. Also, an efficiency value of 38.9 percent is reported on page E-15 of Volume III.

Response: Based on the revised well efficiencies described in response to comment No. 19, the estimated well efficiencies from the Phase I analyses are considered reasonable. Therefore, these well efficiencies will be used for Phase II design. However, the effects of partial penetration and aquifer drainage will also be considered when estimating available drawdown for Phase II extraction wells. As a result, the available drawdown for Phase II design will be less than that calculated based on well efficiency alone. Phase II well efficiency design values, partial penetration effects, and dewatering effects will be considered during Phase II design and discussed in the Phase II Extraction Well Plan. Sections 4.2.3.2 and 4.2.4.2 of the ER were modified to reflect the revised well efficiency estimates and indicate that these values will be used for Phase II design (see Sections 4.2.3.2 and 4.2.4.2 in Attachment B).

21. Comment: There is some apparent contradiction in the description of the Basalt Aquifer, east of the landfill. In Section 5.0 the Basalt aquifer is stated as having an apparent capacity of 5 gpm which limits effectiveness of groundwater extraction for remedial purposes. In Section 4.2.6 private well pumping in the basalt aquifer, east of the landfill, is given as a possible reason for migration of contamination thus indicating a permeable basalt aquifer. We suggest limiting discussion of the flow characteristics of the basalt aquifer to a single section.

Response: A low apparent aquifer capacity or bulk transmissivity for the Basalt Aquifer does not necessarily contradict the conceptual model presented in Section 4.2.6, which suggests that private well pumping is a possible mechanism for constituent distribution east of the Landfill. Although the bulk transmissivity is low for the Basalt Aquifer, the transmissivity for individual fractures or fracture zones may be much higher. High transmissivity, combined with the low storativity that is often characteristic of fracture flow, provides a reasonable mechanism for observed contaminant migration east of the Landfill. This mechanism cannot necessarily be discerned from pumping test data and does not require a high bulk transmissivity to occur. A paragraph was added to Section 4.2.6 to clarify the conditions under which an aquifer can exhibit both low bulk transmissivity and constituent migration over a significant distance (see Section 4.2.6 in Attachment B).

It is not practical to present a conceptual flow model for groundwater infiltration/migration in the vicinity of the Landfill and limit discussion of Basalt Aquifer flow characteristics to a single section. Consequently, revision of the text to limit discussion of Basalt Aquifer flow characteristics is not proposed in response to this comment.

22. Comment: We are unable to verify the conclusion made in section 4.3 that the Upper Sand/Gravel Aquifer, and possibly the shallow interbeds of the Lacustrine Aquitard, recharge the Fluvial Aquifer through springs, and appear to be the source of the Constituents of Concern detected therein. Please reference data or measurements to support the conclusion.

Response: The springs located along the bluff overlooking the Little Spokane River recharging the Fluvial Aquifer was first discussed in Section 4.2.3.1 (page 4-12, fourth paragraph) and reiterates the conclusion drawn in the RI (RI Section 5.2.1.1). Chemical data for selected springs were presented on Figure ER-4.39 and indicate a prolonged history of TCA presence in a number of springs that discharge to the Fluvial Aquifer. Constituent distribution data for the Lower Sand/Gravel Aquifer indicate the Constituents of Concern do not extend as far west as the spring locations (see Figures ER-4.30 through ER-4.35). Consideration of these combined conditions strongly support the conclusion that the Constituents of Concern detected in the Fluvial Aquifer are related to the Upper Sand/Gravel Aquifer and not the Lower Sand/Gravel Aquifer. The reference to the Upper Sand/Gravel Aquifer recharging the Fluvial Aquifer was moved from Section 4.3 to Section 4.3.2.1, which is the more appropriate section for discussing the bases for this conclusion (see Section 4.3.2.1 in Attachment B). Further revision to the ER is not proposed in response to this comment.

23. Comment: Please provide a table showing the density, solubility, partition coefficient, vapor pressure and other pertinent physical and chemical characteristics of the Contaminants of Concern as they relate to their fate and transport in groundwater, in an air stripping tower, and in the atmosphere.

Response: A table (Table ER-4.3) showing the specific gravity, solubility, partition coefficient, retardation factor, vapor pressure, and Henry's constant for the Constituents of Concern was added to Section 4.0 and referenced in Sections 4.3 and 4.4.2. These parameters address physical and chemical properties that affect constituent migration in groundwater and behavior in the stripping tower. We are not aware of additional, generally accepted constituent physical or chemical properties that apply to constituent fate and transport in the atmosphere (see Table ER-4.3 in Attachment B).

24. Comment: There is confusion over the trough in the upper surface of the Lacustrine Unit. In section 4.2 the trough is described as an apparent trough, based on limited data. Reference is made to the vicinity of pilot well CP-S1 on cross section ER. 4.9. But cross section ER 4.9 does not display well CP-S1. After reading Section 4.2 we are not convinced that the trough exists. However, in section 4.3.2.1 a contaminant migration path is said to conform to the north-south tending trough identified in Sections 4.1 and 4.2. Section 4.3.2.1 gives the reader the impression that no question exists as to the presence of the trough. In summary, we are not sure of the reports position regarding the trough.

Response: The existence of the trough is unconfirmed. As such, "apparent" was inserted prior to "north-south" in Section 4.3.2.1. References in the text to Pilot Extraction Well CP-S1 on Figure ER-4.9 were changed to reference Monitoring Well CD-30A.

25. Comment: In section 4.3.2.1 the statement is made that the peak and subsequent decrease in contamination at the Friedrichsen well (shown on Figure 4.38) probably represent a leading edge "stringer" in advance of the main body of the plume, and that concentrations are expected to increase at this location in the future. We do not understand this logic since every one of the last seven samplings of the well have shown a decrease in contamination. Please elaborate on the conclusion that concentrations will increase.

Response: Groundwater quality data for Monitoring Wells CD-30 and CD-33, and Pilot Extraction Well CP-S1 exhibit significantly higher concentrations of TCA and are located approximately 350 ft upgradient of the Friedrichsen well. Similar concentrations will ultimately reach the Friedrichsen well unless intercepted by the Phase II South System. The ER was modified to reference this upgradient water quality data (see Section 4.3.2.1 in Attachment B).

26. Comment: We disagree with the statement made on 4-23 that the TCA concentrations in springs, over time, shown in figure 4.39 suggest a depleted source. We believe the data suggests the source has been reduced, but that it is not depleted.

Response: Although available data suggest that the source is "depleted," but not necessarily exhausted, "depleted" was changed to "reduced" on page 4-23 of the text.

27. Comment: We do not concur with the statement on page 4-23 that TCA concentrations will ultimately decrease in the fluvial aquifer because the lateral extent and thickness of Fluvial (unit) discussed in sections 4.1.1.7 and 4.1.2.7 is unknown, and the springs are only one point of discharge to the aquifer.

Response: The statement on page 4-23 that TCA concentrations will ultimately decrease in the Fluvial Aquifer is based on constituent concentration trends in the Upper Sand/Gravel Aquifer, including springs that directly discharge to the Fluvial Aquifer. These data suggest that constituent concentrations in the Upper Sand/Gravel Aquifer are decreasing in the vicinity of the Landfill and in other areas that may discharge to the Fluvial Aquifer. This conclusion is based on data independent of the lateral extent and thickness of the Fluvial Aquifer, and is a reasonable conclusion based on available data. The text was modified to reference the time versus concentration data in the Landfill vicinity (Figure ER-4.37) as additional support to this conclusion. No other modification to the ER is proposed in response to this comment (see Section 4.3.2.1 in Attachment B).

28. Comment: The migration rate for TCA on page 4-23 is unsubstantiated because no calculations are presented or referenced. Substantiate the rate by identifying the assumptions and displaying the calculations.

Response: All the data needed to estimate TCA migration rate was presented in Table ER-4.3 (revised Table ER-4.5). Formulas for calculation of TCA migration rate and retardation factor were added to revised Table ER-4.5 in response to this comment (see Table ER-4.5 in Attachment B).

29. Comment: Please quantify "minor exceedances" used in Section 4.3.1.1. What numerical value constituents a minor "exceedance"? Also, the minor "exceedances" of TCA and DCE southeast of the landfill are not shown on the figures for Section 4.0 and should be shown.

Response: The term "minor exceedances" was used after introduction of the contaminant distribution data in Figures ER-4.24 through ER-4.29. The text was modified to provide references to appropriate figures when qualifying exceedances with adjectives such as "minor". However, it will be necessary for the reader to examine the figures to put adjectives such as "minor" and "significant" into context (see Sections 4.3.1.1 and 4.3.1.2 in Attachment B).

The minor exceedances of TCA and DCE southeast of the Landfill refer to Well CD-23 and are shown on Figures ER-4.24 and ER-4.25, respectively. No modification to the ER is proposed in response to this comment.

30. Comment: In Section 4.3.2.2 the single reference to landfill disposal history as a means for explaining contaminant migration is not acceptable. If landfill history is to be used for an explanation then state specifically how the history is a factor and provide the history for the reader. If reference is made to County records provide a copy of the records or a specific reference. We will attach little significance to conclusions based on evidence not provided for our files.

Response: The RI (RI Section 5.4.1) indicates that solvents were disposed of by pouring the solvent mixtures in trenches that were covered soon after disposal. This method of disposal results in a dispersed source and, when combined with a limited source volume and thick vadose zone (greater than 40 ft), does not provide the source mechanism typically required for accumulation of large masses of DNAPL at a single location or for migration of DNAPL a significant distance from the Landfill. Reference to the RI as the source for this disposal history, and a clarification of how this disposal history impacts DNAPL migration (as described herein), was incorporated into the text of the ER. No other modification to the ER is proposed in response to this comment.

31. Comment: In Section 4.3.3 when using landfill history to support assumptions please follow the procedures in the above comment.

Response: Section 1.3 of the RI indicates that Key Tronic disposed of both TCA and methylene chloride at a rate of several hundred gallons per month, and that this disposal occurred from 1975 to 1980. The text of the ER was modified to reflect this reference. The text was also modified to indicate that the discrepancy (between estimated masses of TCA and methylene chloride in the groundwater flow system) may be the result of overestimating the methylene chloride disposed mass or underestimating the TCA disposed mass (see Section 4.3.3 in Attachment B).

32. Comment: In Section 4.3.2.2 the reference to constituent concentrations decreasing to a level significantly below that which would be expected is not acceptable as evidence for the absence of DNAPLs. Please identify what levels would be expected, and explain why the observed levels deviate from expected levels.

Response: Constituent concentrations of about 1 percent or more of the solubility limit of the constituent would be expected for groundwater in the vicinity a DNAPL pool. This would translate to a TCA concentration of approximately 9,000 ppb, significantly higher than the highest concentration (39 ppb) detected during Phase I in the Upper Sand/Gravel Aquifer in the Landfill vicinity. Section 4.3.2.2 was modified to present this comparison of expected concentration (if DNAPL was present) versus observed constituent concentrations (see Section 4.3.2.2 in Attachment B).

The comment also asks for an explanation of why the observed levels deviate from the levels expected (if DNAPLs were present). The comparison of expected to observed concentrations was presented in the ER as part of the data that suggest that DNAPLs are not present in the groundwater flow system at the site. There does not appear to be a need for further explanation within this context. Consequently, modification to the ER text beyond that described in the previous paragraph is not proposed in response to this comment.

33. Comment: In Section 4.3.2.2 the mention of constituent concentrations decreasing significantly is not sufficient to support the absence of DNAPLs. Explain significance. What other reasons for the decreasing levels were rejected before arriving at the absence of DNAPLs as the best explanation?

Response: Dissolved constituent concentrations in the vicinity, and downgradient, of a DNAPL source would be expected to remain high for tens of years because of the relatively low water solubility of most chlorinated solvents. As a result, the observed significant decrease in groundwater concentrations over a relatively short period of time (10 years) suggests the absence of a DNAPL source in the vicinity of a given monitoring location. The text in Section 4.3.2.2 was expanded to include this explanation (see Section 4.3.2.2 in Attachment B).

The comment also asks what other reasons for the decreasing levels (in concentration) were rejected before arriving at the absence of DNAPLs as the best explanation. It should be recognized that data were evaluated during Phase I without a preconceived mechanism or explanation for observed conditions. Therefore, the presence of DNAPLs was considered as a possible explanation for anomalous constituent distribution characteristics, but was not assumed to be the correct explanation. As such, conditions that would allow a decrease in groundwater concentration in the vicinity of a DNAPL source were not further considered.

34. Comment: In Section 4.3.2.2 what other reasons for the distribution of contaminants were rejected before arriving at the absence of DNAPLs as the best explanation for the distribution?

    Response: As discussed in response to comment No. 33, it was not assumed that contaminant distribution east of the Colbert Landfill was the result of DNAPL migration. Therefore, DNAPLs were considered as a possible explanation for the horizontal and vertical distribution of the Constituents of Concern, along with the other possible mechanisms discussed within the ER. However, DNAPL migration was rejected as a probable explanation for observed migration patterns because little or no evidence was obtained that support this hypothesis. Revision to the ER is not proposed in response to this comment.

35. Comment: In Section 4.3.2.2 please reference the low permeability contact with a cross section so the reader can find it.

    Response: The ER text was revised to refer the reader to Geologic Cross Section C-C' (Figure ER-4.5). Additionally, the referenced contact was changed from the Lower Sand/Gravel-Latah Formation interface to the Upper and/or Lower Sand/Gravel-Latah Formation interface because the distinction between the Upper and Lower Sand/Gravel Aquifers is unimportant and difficult to discern east of the Landfill, where the Laustrine Aquitard is not present (see Section 4.3.2.2 in Attachment B).

General Response to Ecology Comments Nos. 30 and 32 through 35: The commenter is correct in suggesting that when considered individually, data presented in Section 4.3.2.2 are insufficient for concluding that the observed anomalous constituent migration patterns do not appear to be the result of DNAPL migration. However, when these data are considered together, they strongly suggest that DNAPL migration is not the cause of observed anomalous constituent migration.

The County is willing to modify this conclusion if data are provided that indicate such a modification is warranted. However, the available data more strongly support the ER conclusion that DNAPLs are not present in the groundwater flow system, than the RI conclusion (that they are). Modification of the ER on this issue (beyond the modification previously described) is not proposed.

36. Comment: In Section 4.3.2.2 please provide evidence or support for the statement that contaminant migration in the lower aquifers east of the landfill will revert to directions consistent with groundwater flow when no longer influenced by private pumping.

Response: This statement is an opinion based on available data. However, this statement cannot be validated or invalidated until private well pumping is ceased in the area of concern, and subsequent groundwater monitoring is conducted for an extended period of time (years). We propose to leave the statement in the ER, as it provides useful information on anticipated aquifer response. No modification to the ER is proposed in response to this comment.

37. Comment: The estimate in table 4.4 can not be verified. Explain how the estimate were compiled. Please submit the calculations.

Please submit the calculations for the estimate of flow velocity in table ER-4.2.

In regard to the groundwater monitoring in Section 5.2, what type of monitoring is envisioned to evaluate the performance of the East system?

Response: The procedures used for contaminant mass estimates are provided in Appendix G of the ER. Appendix G is now referenced in Section 4.3.3 and in Table ER-4.6.

All the data needed to estimate flow velocity are presented in Table ER-4.2. The equation for estimated flow velocity (average linear velocity) was added to Table ER-4.2 to provide additional documentation (see Table ER-4.2 in Attachment B).

The scope of work for the Consent Decree specifies that the East System is a source control system (rather than an interception system) and performance monitoring is not required unless monitoring wells upgradient of, and outside the zone of capture for, the East System show a consistent rise in constituent concentration. Data generated by the Domestic Well Monitoring Program and data collected from existing monitoring wells, will be used to assess system performance. Monitoring for the Phase II East System will be addressed in the Phase II Groundwater Monitoring Plan. No modification to the ER is proposed in response to this comment.

38. Comment: The statement in section 4.3.3 that "significant masses of TCA and MC may remain in DNAPL form in the landfill refuge and in the vadose zone underlying the landfill" is not supported by any direct investigation of the refuge for such material. The means for supporting this statement are largely opinions dispersed throughout the report

that can not be substantiated or verified. Also, sampling of the landfill revealed no DNAPL's. We do not accept this statement.

Response: The physical processes that control DNAPL migration in the subsurface support the conclusion that residual DNAPL will remain in interstitial pores in the soil matrix from the DNAPL source to its maximum extent of migration. Thus, if DNAPLs are present anywhere in the Landfill vicinity, they are present within the refuse and/or the underlying vadose zone. It is unclear why the commenter would reject this conclusion, yet apparently maintain that DNAPLs are the cause of anomalous contaminant migration to the east of the Landfill (as indicated by comments No. 30, and 32 through 34). No revision to the ER is proposed in response to this comment.

39. Comment: If it is important to determine whether the Upper Sand\Gravel Aquifer and the Lower Sand Gravel Aquifer are recharged by a common source than we suggest plotting three more wells from the Upper Sand\Gravel Aquifer in addition to the two wells plotted on the piper diagram in figure ER 4.45.

Response: At present, it does not appear necessary to determine whether the Upper Sand/Gravel and Lower Sand/Gravel Aquifers are recharged by a common source. Should such a determination become necessary, appropriate data from additional wells will be obtained and plotted on a piper diagram. No revision to the text is proposed in response to this comment.

40. Comment: We do not understand the concept being developed in the last paragraph of Section 4.3.1.2. What conclusion(s) are to be drawn from the paragraph?

Response: The primary purpose of this paragraph was to characterize the vertical distribution of Constituents of Concern in the Lower Sand/Gravel Aquifer, similar to the previous discussion in the text regarding the horizontal distribution of constituents. It can also be concluded from the data presented in this paragraph that constituent migration in the Lower Sand/Gravel Aquifer conforms to groundwater flow and does not exhibit patterns that are indicative of DNAPL presence (as is pointed out in Section 4.3.2.2). No revision to the ER is proposed in response to this comment.

41. Comment: The basis for much of the discussion on contaminant migration in Section 4.3.2.2 is a reference to pumping from private wells as causing anomalous constituent migration observed in the Lower Aquifers. Reference is made to a description of the pumping "mechanism" in Section 4.2.6. However, Section 4.2.6 does not describe the mechanism. Section 4.2.6 merely lists pumping from private wells as one mechanism for a probable cause. We find no support for the conclusion that pumping from private wells has an impact on contaminant migration, and therefore, cannot concur with the conclusion.

Response: As discussed in response to comment No. 21, the mechanism for upgradient and cross-gradient migration of Constituents of Concern in the Basalt Aquifer includes localized gradient reversal and migration of constituents along individual fractures or within fracture zones. A statement to this effect was incorporated into Section 4.2.6. It is recognized that direct evidence of domestic well pumping impacting contaminant migration does not exist. Because of this, it was identified as a potential mechanism (along with migration along an unidentified extension of the Laustrine Aquitard), not a confirmed mechanism. We believe that the two mechanisms provided in the ER (private well pumping and migration along low permeability contact) represent the most probable mechanisms for causing the anomalous constituent distribution east of the Landfill, based on available data. As such, modification to the ER, beyond that previously described to clarify the private well pumping mechanism, is not proposed in response to this comment.

42. Comment: On page E-15 of Volume III, explain the rationale for selecting the saturated thickness of 17 feet to estimate K for the Upper Sand\Gravel Aquifer. Please reference water level measurements and geologic logs.

Response: The estimated saturated thickness used to estimate K was based on a preliminary geologic profile for the CP-S1 borehole and a measured depth to water of 89 ft. Boring logs In Appendix B were modified to display approximate depth to water so that saturated thickness can be easily approximated from the boring logs, and boring logs are now referenced, where appropriate, within the text of Appendix E (see Appendix E in Attachment C).

Upon review of the data for Pilot Extraction Well CP-S1, it was determined that geologic unit contacts were modified slightly between the preliminary and final version of the boring log, and a more appropriate saturated thickness in the Extraction Well CP-S1 vicinity is 19 ft. The revised saturated thickness estimate and associated K estimate are reflected in Appendix E (see Attachment C). The associated estimates of average linear velocity in the main body of the ER were also modified to reflect the revised estimate of saturated thickness (see Table ER-4.2 in Attachment B).

43. Comment: On page E-15, explain why a geometric mean was used for T and K instead of an arithmetic mean. Was the data log normally distributed and how did you make the determination: Table E-1 shows an average value, not a geometric mean.

Response: An arithmetic mean was used to estimate T and K, and was erroneously reported as a geometric mean in the ER. The ER was revised to reflect an arithmetic mean (see Section 3.1 in Attachment C).

44. Comment: On page E-16 an average value is reported for Sy. Is this an arithmetic mean? If so, why is an arithmetic mean used on one set of aquifer parameters and a geometric mean on another?

Response: The average value report for Sy is an arithmetic mean. This is consistent with the statistical approach used for determining average values of T and K described in response to comment No. 43. No revision to the ER is proposed in response to this comment.

45. Comment: Where is the distance drawn down plot discussed in the first paragraph on page E-16? Please show the calculations for estimating the radius of influence.

Response: The distance-drawdown plot for the CP-S1 APT was provided on Figure E-10 and is now referenced in the Appendix E text.

The estimation of radius of influence is not a calculation, but a best fit straight line projection through the distance-drawdown data to an intercept of 0 drawdown (r_o). Appendix E Section 2.1 (Method 5) was modified to describe the method for estimation of radius of influence (see Section 2.5 in Attachment C).

46. Comment: What steps were taken to insure the transducers were operating properly? Were any measurements taken with an E-tape or tape measure to substantiate the transducer measurements for CP-W1 and CP-S1?

Response: As stated in Section 3.4 of the ER (page 3-5), groundwater levels were collected manually from wells monitored by data loggers for calibration purposes using electric sounding tapes. Drawdown versus time data for hand level measurements were plotted on a computer printout of drawdown versus time generated from data logger data to confirm the accuracy of the electronically collected data. The manually collected water level data are not presented in the ER, but are maintained in Landau Associates' files and are available for review. No revision to the text is proposed in response to this comment.

47. Comment: On page E-17 please explain the rationale for selecting 175 feet as the saturated thickness.

Response: The 175-ft saturated thickness used for estimating hydraulic conductivity was based on the Lower Sand/Gravel Aquifer saturated thickness from Monitoring Well CD-47 C2 geologic profile (i.e., the difference in depth between the top and bottom of the aquifer). CD-47 is the closest well location to CP-W1 where a boring was extended to the base of the Lower Sand/Gravel Aquifer. The text on page E-17 was modified to reflect this information. It should be noted that this saturated thickness value differs slightly from that provided on Figure ER-4.17 for Well CD-47 (177 ft). This difference is due either to round-off error or the use of preliminary boring logs for development of Figure ER-4.17, but the difference (1.1 percent) is considered inconsequential for subsequent use. The value for saturated thickness at Well CD-47 was modified on Figure ER-4.17 to conform to that obtained from the boring log in Appendix B of the ER.

48. Comment: In analyses of both tests why is emphasis placed on an upward bound for T? Should not a lower bound be considered in the design process in order to consider the possibility of de-watering the aquifer?

Response: As discussed in response to comment No. 15, we concur that consideration of a lower bound value for transmissivity is also important, particularly for the Upper Sand/Gravel Aquifer because of its limited saturated thickness. Lower bound transmissivities for CP-S1, CP-W1, and CP-E1 are 10,000 ft^2/day, 30,000 ft^2/day, and 10,000 ft^2/day, respectively. These lower bound estimates were based on review of the data and professional judgment. Reasonable lower bounds are presented in revised text to Appendix E and the main body of the ER for the Upper and Lower Sand/Gravel Aquifers (see appropriate sections of Attachments B and C).

49. Comment: Please describe to what extent the wells recovered. Is the recovery sufficient for the test to be valid?

Response: Recovery data were collected until the pumping well and primary observation wells recovered to at least 90 percent of their prepumping water level. We believe that adequate recovery data were collected for the tests to be valid. No modification to the ER is proposed in response to this comment.

50. Comment: Borehole CD-4 is a pivotal borehole for several cross sections and for the conceptual model of groundwater flow shown in Figure 4.23. However, borehole CD-4 was installed with an air rotary rig by a consultant not affiliated with Phase I. In light of the reduced control over the lithology of borehole CD-4 what is the confidence level for hydrogeological interpretations based on CD-4?

Response: As stated in the ER, boring logs based on cutting samples from air rotary drilling do not provide the level of detail obtained from driven samples collected during cable tool drilling. However, the geologic data obtained using air rotary drilling should be sufficiently accurate to identify major geologic contacts. The elevations of major contacts identified on the borings log for Well CD-4 are consistent with elevation trends for geologic contacts identified in nearby Phase I boring logs. As a result, the accuracy of the geologic information for Well CD-4 appears adequate for the purposes to which it was applied during Phase I analyses. No modification to the ER is proposed in response to this comment.

51. Comment: Information is presented and discussed in this report that is taken from borehole logs not contained in the report. All borehole logs used in this report should be referenced so that a reader not intimately familiar with the project can find them.

Response: The text for Section 2.1.1 was modified to refer the reader to the appropriate appendices of the RI for boring logs of wells installed by Golder Associates and Maddox & Associates.

52. Comment: This document is a draft report but is not labeled as such.

Response: The revised Phase I Engineering Report that incorporates revisions resulting from EPA and Ecology comments will be labeled "Final" to differentiate between the final and draft reports.

53. Comment: The scale accumulation in the well pump and stripping tower pose major problems because the system was in operation for only a few days and remediation will most likely take years. Both permitting and logistical problems may arise if acid

treatment is selected as a remedy. This problem should be addressed early in the Phase II design.

A monitoring system will most likely have to be installed to prevent critical buildup of scale. Conceptually, the Phase I Report should describe how the scale buildup will be dealt with.

Response: We concur that scale control is an important consideration for Phase II design and should be addressed early in the design process. Scale control will be addressed in the preliminary Phase II Treatment and Discharge Plan and subsequent submittals.

It is unclear why acid treatment would necessarily cause permitting and logistical problems for Phase II operation. If acid is used for pH control, discharges will be within the pH limits generally acceptable for surface water discharges. If acid is used for batch treatment, certain manifesting, containment, handling, and disposal practices must be employed. But, batch treatment should not present a permitting difficulty, particularly if the treatment is contracted to an independent company (as is envisioned). The logistics for acid treatment require consideration, but do not appear to present any problems that have not been addressed on numerous other projects.

An on-line scale monitoring system will be considered during the Phase II design, and will be evaluated based on system reliability, accuracy, and cost-effectiveness. Access ports will be provided for visual inspection of packing, and a mass balance of alkalinity based on influent and effluent concentrations will provide a method of monitoring scale accumulation. Section 4.4.6.3 discusses the options for Phase II scale control. Further evaluation of these options and selection of the most appropriate scale control method will be accomplished as part of Phase II design. Modification to the ER is not proposed in response to this comment.

54. Comment: Monitoring the performance of remediation as well as the impact of pumping on local aquifers and the Little Spokane River is critical. Conceptually, the Phase I report should describe how monitoring of the performance and impact will be dealt with.

Response: We concur that monitoring the performance of the remediation as well as the impact of pumping on local aquifers is important. However, the basis for monitoring of interception systems and long-term compliance monitoring is provided in the Consent Decree Scope of Work, and restatement of these performance criteria in the Phase I Engineering Report does not appear warranted. Specific monitoring components will be presented in Phase II work plans and in the Phase II operations and maintenance plan, providing EPA and Ecology an opportunity to comment on proposed procedures. No modification to the ER is proposed in response to this comment.

55. Comment: No mention is made for groundwater monitoring down gradient of the East system. What is the conceptual plan for monitoring the performance of the East system?

Response: The Consent Decree Scope of Work specifies that downgradient performance monitoring for the East Extraction System is not required. The response to comment

No. 37 discusses other aspects of Phase II East System monitoring. No modification to the ER is proposed in response to this comment.

56. Comment: Paragraph 4, page 4-28, the test indicates that the influent air temperature range experienced during the study represent the lower range of operating conditions for the full scale system, but that these are not the lowest temperatures anticipated. Does this imply that at lower temperatures the system may not be operated?

Response: The referenced statement is not meant to imply that the system will not operate at lower temperatures. The referenced paragraph goes on to state that even at subzero air temperatures, the water temperature remains relatively constant throughout the tower. This suggests that the tower can operate at substantially lower air temperatures than observed during Phase I. No revision to the ER is proposed in response to this comment.

57. Comment: Paragraph 2, page 4-32, figures ER-4.46, ER-4.47, and ER-4.48 show data fit lines produced on the basis of small numbers of data points which are tightly clustered at two hydraulic loading rates. Most of these lines are essentially linear regressions performed on two points, and should not be assumed to accurately predict the removal efficiency of the treatment system between the clusters of data points.

Response: The primary purpose of these graphs is to demonstrate the relatively small changes in removal efficiency with hydraulic loading for both the 2-inch and 3.5-inch packing materials, and the higher removal efficiency for the 2-inch packing than the 3.5-inch packing. The best fit lines shown on the figures were for representational purposes only and were not used for interpolation between data points. No modification to the ER is proposed in response to this comment.

58. Comment: Paragraph 2, page 4-33, while running counter to the theoretical behavior of air stripper systems as described in this section, Figures ER-4.49 and ER-4.50 might be reasonably interpreted as showing a nonlinear response to an increase in air/water ration. They seem to display a general decrease in removal efficiency for air/water ratios in the range of 70-75, with an increase in efficiency as the ration moves above 75.

Response: Although nonlinear response is quite possible, an actual decrease in removal efficiency with increasing air/water ratio is improbable. It is more likely that the apparent decrease in removal efficiency is the result of data scatter resulting from variations in operational conditions such as influent concentration. Because of the apparent data scatter, interpretations beyond the general trends in removal efficiency with increasing air/water ratio were not incorporated into the ER. No modification to the ER is proposed in response to this comment.

59. Comment: Paragraph 3, page 4-31, the text states that the average influent and effluent concentrations of methylene chloride are presented in Table ER-4.6. These concentrations appear in Table ER-4.7.

Response: The text was modified to reflect that average methylene chloride values are presented in Table ER-4.7 (revised Table ER-4.9).

60. Comment: Paragraph 1, page 4-346 [sic], the relationship between hydraulic loading and tower diameter should be explained. While the effects of varying hydraulic loading on treatment performance are discussed in Section 4.4.4.a [sic], the effects on tower dimensions are not.

Response: The tower diameter is selected to achieve a design hydraulic loading in terms of gpm/ft^2 or other mass flux per unit area units. As such, tower diameter is not a primary design parameter, but is a dependent variable selected based on the design hydraulic loading (similar to the blower capacity being dependent on the design air:water ratio for a given hydraulic loading). The dependence of tower diameter on hydraulic loading was briefly addressed in the first sentence in Section 4.4.6.2. This sentence was modified to state the dependence of tower diameter on hydraulic loading. No other modification to the text is proposed in response to this comment (see Section 4.4.6.2 in Attachment B).

61. Comment: Paragraph 4, page 4-37: If different designs of packing materials are to be evaluated, that evaluation should have been one of the objectives of the treatability study. The two packing materials used in the study apparently are of similar design, and differ primarily in diameter and surface area. It should be explained in some detail how the empirical data about these two materials will prove useful in evaluating the published properties of packings with substantially different designs.

Response: During the pilot study, performance characteristics were evaluated for the packing tested, with analysis of the Treatability Study data using the model equations developed by Onda (the VOLSTRIP model). The Treatability Study data and published mass transfer coefficients for the tested packing will be compared to published performance data for alternative packings. The published mass transfer coefficients will be adjusted using the Treatability Study results (adjustment factors) prior to use in the model equations to predict their performance. In addition, case study operational data will be evaluated (if available) for alternative packings to adjust published data. Section 5.6.3 was added to the ER (in part) to describe how Treatability Study data will be used during Phase II design (see Attachment A).

62. Comment: Appendix G-Example Calculations, page g-3, the origin of the Aquifer TCA Volume is not clear. The units on the term appear to indicate that it is the average concentration of TCA in a column with dimensions of unit area and thickness of the saturated layer. If this is the case, the example calculations do not illustrate all the steps outline in paragraph 5 of this section.

Response: Aquifer TCA volume is calculated by Geographic Information System (GIS) software, and represents the average cell concentration times the average saturated thickness for a given aquifer unit. This value is then multiplied by the average cell area and the aquifer porosity to calculate the Constituent Water Volume. The method by which this value (Aquifer TCA Volume) is calculated will be provided in the calculation section. However, specific values cannot be provided for average cell concentration and average saturated thickness, as they are for the other calculated values (TCA Water Volume, TCA Mass, and TCA Volume), because these values are not provided as output by the GIS.

63. Comment: Appendix H-Table 2, the outputs listed do not include all the outputs shown in the results of modeling. On page 6 of the output, the column labeled "Z (ft)" is not explained, nor is the safety factor defined in terms of what is signifies.

Response: The value "Z" represents the packing height required for change from specified influent to specified effluent concentration. This value is represented by $h_{t,i}$ in Table 2. Table 2 was modified to indicate that $h_{t,i}$ is represented by "Z" in the output. The safety factor represents the excess height of packing beyond that which is required to meet the effluent concentration for a given constituent. The equations used to calculate SF and $h_{t,i}$ are presented as Equations 31 and 32 of the VOLSTRIP Model Governing Equations section that follows Table 2. A percent symbol was added to Table 2 following the safety factor description to denote output units. Additional description of $h_{t,i}$ and SF are not proposed in response to this comment.

It should be noted that on page 6 of each model output, the Z and SF values for compound No. 2 (1,1-DCA) are negative. This is the result of the influent concentration being below the Performance Standards and is not an indication of inadequate performance.

64. Comment: The Phase I activities regarding air modeling are in agreement with previously established protocols.

Response: This comment is interpreted to signify that the air emissions assessment is adequate and Phase II offgas emissions will not be required provided the assumptions used to develop the air quality model are confirmed to be accurate (or conservative) during Phase II design.

RESPONSE TO E&E SEPTEMBER 9, 1991 COMMENTS

ANALYSES

1. Comment: During Phase I, methylene chloride data were qualified using two different sets of criteria to account for laboratory blank contamination. The first set of criteria was based on United States Environmental Protection Agency (EPA) National Functional Guidelines. All methylene chloride values reported that were at levels below 10 times the amount found in the associated blank were qualified as undetected ("U"). Apparently, a second set of criteria was implemented for using the methylene chloride data in the treatability study. These criteria involved blank corrections for all sample results where the reported level of methylene chloride was about two times the level found in the associated blank sample. Reported levels of methylene chloride in samples that were between two and five times the level found in the associated blank were qualified as "B" after blank correction, while reported levels greater than five times the

amount found in the associated blank were not qualified after blank correction. Results below two times the amount found in the associated blank were qualified as undetected ("U").

Blank correction is not an EPA-accepted practice, as errors associated with low level measurements may be compounded. The use of blank correction for methylene chloride data in the treatability study resulted in reported low levels of methylene chloride where the use of EPA Guidelines would have resulted in reported elevated quantitation limits reported as undetected ("U").

The following methylene chloride sample results were reported as detected for the treatability study but were reported as undetected using EPA guidelines: [Table deleted in this Response to Comment for the purposes of brevity]

The values were taken from Table D-3 of the Colbert Landfill Phase I Data Validation Report, Appendix D. By using two sets of criteria, two sets of data are generated. This may lead to confusion for future data users, and result in qualified data that is used for the wrong purpose. Comparison of these results with those presented in Table ER-3.6 of the ER was not readily made due to differing sample identifiers.

Response: It is recognized that blank correction may sometimes lead to errors associated with low level measurements, and as a result methylene chloride data were validated using EPA Functional Guidelines for overall data validation purposes. However, methylene chloride laboratory contamination is ubiquitous and applying EPA Functional Guidelines to the data utilized for treatability study analyses would result in a large percentage of the data being unusable. Elimination of data from the treatability study due to elevated quantitation limits (using EPA Functional Guidelines) would have compromised the treatability study analyses to a greater extent than that caused using blank-corrected values. Blank correction is a practical method of addressing laboratory contamination, provided the limitations of the data are recognized.

To minimize the potential confusion created by the presence of two data sets for methylene chloride, text was added to Section 4.4.1 identifying that the data in Table ER-4.6 were blank corrected for the purposes of treatability study analyses and referring the reader to the analytical results in Appendix F for data validated in conformance with EPA Functional Guidelines (see Section 4.4.1 in Attachment B).

The data presented in Table ER-3.6 were not blank corrected and were validated in conformance with EPA Functional Guidelines.

2. Comment: Tables D-4 and D-5 are missing from Appendix D of the Colbert Landfill Report Phase I (data validation section).

Response: Copies of Tables D-4 and D-5 were transmitted to EPA and Ecology subsequent to receipt of draft E&E comments on September 13, 1991. These tables will be incorporated into the Final ER.

3. Comment: Typographical errors were found in Table D-3 (i.e., sample FB-1/31/91 EPA Data Validation Guideline value should have an associated "U" qualifier, and sample FB-3/19/91 Treatability Study value should have an associated "U" qualifier).

Response: A "U" qualifier was added to samples FB-1/31/91 and FB-3/19/91 for the EPA Data Validation values in Table D-3.

TREATABILITY/DESIGN

1. Comment: The report concludes that air stripping can achieve removal efficiencies necessary to meet Performance Standards for Phase II effluent discharge based on methylene chloride as the critical constituent affecting design. It is noteworthy that of the 16 treatability test trials several approached the Methylene Chloride Performance Standard but only one trial was below the 2.5 ppb limit. The report concludes that the effluent standards could be achieved by increasing the tower packing height. We concur with this conclusion. The Phase II treatment system design should now incorporate the findings of the Phase I design to complete the treatment system design.

Response: Phase II effluent standards may also be achieved with a tower of equal or lesser height because Phase II groundwater extraction in areas where methylene chloride is not present in significant concentration is expected to result in lower methylene chloride influent concentrations than were observed during Phase I. Modification to the ER is not proposed in response to this comment.

2. Comment: Chemical scale control is noted as a key design consideration based on problems encountered during treatability studies. The report states that the method of scale control will be selected in the Phase II design. Bench-scale studies are highly recommended. Any such information should be included in the Phase II design.

Also, biological growth was identified as a problem in the extraction well system. The possibility of biological growth in the treatment system also should be considered during Phase II design.

Response: It is probable that bench-scale studies will be accomplished for selected scale control method(s). However, the need for, and scope of, any bench-scale studies cannot be determined until Phase II design is initiated. Results from such studies (if implemented) will be provided for review by EPA and Ecology as part of the Phase II design submittal process.

Although biological growth was identified as a potential source of the problem experienced with Pilot Extraction Well CP-E1, no evidence of biological growth was observed in the stripping tower during Phase I. Although the possibility for biological growth will be considered during Phase II design, it is anticipated that consideration will be limited to incorporating access for potential add-on treatment to address biological growth should such growth develop during Phase II operation. No modification to the ER is proposed in response to this comment.

3. Comment: Other design considerations that may be applicable to the Colbert treatment system design are:

- o Aesthetics in relation to the surrounding area;
- o Noise control;
- o Prevailing wind patterns (location of air intake to avoid short-circuiting between the effluent air);
- o Water distribution (to avoid side-wall effects)--this was a design deficiency in the pilot study that was addressed;
- o Mist elimination;
- o Peak flow considerations;
- o Scale-up considerations (i.e., from pilot-scale results to full-scale design);
- o Additional design information on tower flooding considerations for full-scale design; and
- o Final results of air discharge modeling/design, determining the need for air pollution control devices.

Response: We concur that the listed design considerations may be applicable to Phase II design. The extent to which these design considerations are applicable will be evaluated during Phase II design, and will be addressed (as applicable) in Phase II design documents. No modification to the ER is proposed in response to this comment.

4. Comment: The report indicates that the designer expects a time-lag between the extraction system and treatment system designs. This will allow additional time for detailed treatment system design and would result in a more cost-effective design. Whereas this may be valid, E & E did not evaluate whether the Consent Decree allows this scheduling flexibility.

Response: As stated in response to Ecology comment No. 3, it is not yet known whether a modification to the schedule for Phase II design submittals is needed. If such a need is identified, a request for schedule modification will be submitted to EPA and Ecology. There does not appear to be any provisions in the Consent Decree that would preclude such as modification, provided it is accomplished with the concurrence of EPA and Ecology. No modification to the ER is proposed in response to this comment.

5. Comment: The Phase II Design should present details on hydraulic considerations associated with pumping from three extraction systems to one stripping tower.

Response: Hydraulic considerations such as back flow prevention, flow equalization, mixing, and manifolding will be addressed as part of Phase II design. Additional considerations (if any) should be provided by the commenter for consideration during the Phase II design process. No modification to the ER is proposed in response to this comment.

6. Comment: With the change in design to one air stripping tower, flow and mass loading the Little Spokane River will be increased. This may require additional/new permits or approvals from the governing regulatory authority (e.g., State of Washington). There could be a time-lag associated with obtaining such approval.

   Response: As stated in response to Ecology comment No. 5, Subsection 5.6.4 discusses environmental permitting requirements, and was incorporated into the ER (see Section 5.6.4 in Attachment A).

7. Comment: Page 5-1 of the conclusions and recommendations notes several deviations from the ROD.

   Response: The ROD provides conceptual designs for the South, West, and East Systems. It is our understanding that the modification of well and/or treatment facility locations from those provided in the ROD conceptual design does not constitute deviation from the ROD, provided the performance goals of the Remedial Action are achieved. However, we understand it is EPA's responsibility to assess the conformance of the conclusions and recommendations provided in the ER with the requirements of the ROD. No modification to the ER is proposed in response to this comment (see EPA verbal comment and associated response on page 28 of this letter).

8. Comment: The Phase II design should include continued air contaminant dispersion modeling. Federal Clean Air Act revisions and emerging state and regional air quality regulations could impact this design.

   Response: The bases for determining the need for air emissions abatement is provided in Section V.D. of the Consent Decree Scope of Work. Additional air dispersion modeling will be accomplished (if needed) to determine whether air emissions abatement is required in conformance with the bases provided therein. It is our understanding that the bases for design, construction, and operation of the remedial action are the applicable federal laws in effect at the time the Consent Decree was signed. As such, emerging state and federal laws should not impact Phase II design. No modification to the ER is proposed in response to this comment.

GEOLOGY/HYDROGEOLOGY COMMENTS

1. Comment: Proposed modifications to the placement and design of the east and west groundwater extraction systems previously defined in the Scope of Work are based, in part, on a hydrogeologic conceptual model of the Lower Aquifer, as presented in Figure 4.22 of the Phase I ER. The model identifies an apparent groundwater divide in the immediate proximity of the landfill. E & E supports the model, but would suggest that the significance of the condition not be underemphasized. The geology and

hydrogeology beneath the landfill and to the immediate north, northeast, and east are complex and not clearly understood.

Geologic data indicate that the divide is caused, in part, by a Latah Formation structural high, creating a "lobe of Latah Aquitard" protruding westward under the landfill (see Figure ER 4.8). Preliminary isopach mapping of the overlying Lower Sand/Gravel Unit indicates that thinning has occurred over the feature.

It appears that deposition of the Lower Sand/Gravel Unit as well as the Weathered Upper Latah Subunit were influenced by the Latah feature.

The Latah structure may be a more pronounced east-west trending feature than presently identified, and also may be characterized by significant south dip components which could be locally controlling Lower Aquifer contaminant migration toward the north-northeast. If so, local groundwater flow paths within the Sand/Gravel Unit, Weathered Latah Subunit, and/or Basalt Unit may actually include movement toward the north-northeast before swinging westward into the north-south trending valley fill of the Lower Sand/Gravel Unit. Also, as evidenced by analytical chemistry data from monitoring wells such as the CD-23 cluster, structural and erosional conditions may be creating a more direct southward component of contaminant transport within the Lower Aquifer, thus also carrying pollutants from the landfill directly southward before swinging to the west.

As stated in the ER, historic and ongoing domestic well usage east and northeast of the landfill has most likely caused some degree of influence on flow paths in that area. The ER suggests that recent decreases in constituent concentrations east of the landfill (see Section 4.3.2.2) support this assessment. It should be noted that constituent concentrations have decreased throughout the monitoring network in recent years (see Figures 4.37 - 4.42) due to an apparent decline in source contaminant influx.

As recommended in the ER, reduction in domestic well pumping near the east and northeast plume boundaries is appropriate, but the Phase II actions also will benefit if plans incorporate a hydrogeologic drilling and monitoring strategy to obtain additional data on subsurface conditions in this area. No such plans were mentioned in the conclusions and recommendations section of the ER.

<u>Response</u>: We concur that flow paths within the Sand/Gravel Unit, Weathered Latah Subunit, and Basalt Unit to the east of the Landfill may include localized flow components that convey contaminated groundwater to the north-northeast or south-southeast of the Landfill. However, locating specific migration paths in these directions does not appear to be practicable due to the erratic nature and limited saturated thickness anticipated for these potential migration paths. It is possible that once Phase II remediation is initiated and large-scale, long-term pumping of the Lower Sand/Gravel Aquifer is implemented, specific areas of interest or concern in the Lower Aquifers to the east of the Landfill will be identified and can be further evaluated. As a result, specific hydrogeologic drilling and monitoring in the Lower Aquifers (except for the Lower Sand/Gravel Aquifer) is not proposed prior to the initiation of Phase II remediation. Monitoring of existing wells in the Lower Aquifers will be addressed in the Phase II Groundwater Monitoring Plan, as will the bases for determining whether additional

hydrogeologic evaluation is needed. Modification to the ER is not proposed in response to this comment.

2. Comment: Aquifer parameter tests on the Basalt Aquifer indicated slow recharge characteristics (see Section 4.2.5, page 4-18) which may not be adequate for extraction purposes. Geologic cross section and well log data suggest that the Weathered Latah Subunit is apparently present east and north of the landfill. Further evaluation and discussion may be appropriate to determine if the Weathered Latah Subunit has been tested adequately for the presence of contaminants and for consideration as an extraction unit.

   Response: Review of the geologic profiles for borings that intersect the Weathered Latah Subunit indicate that the unit consists predominantly of silt and clay deposits containing basalt gravel, although the unit consists of clayey gravel deposits at some locations. Additionally, piezometric levels in the Weathered Latah Subunit are such that available head for groundwater extraction purposes is very limited. The combination of low permeability and limited available head indicate that this unit would not be appropriate for groundwater extraction. Should data collected during Phase II Remedial Action indicate that this conclusion is erroneous, the Weathered Latah Subunit will be reconsidered as a potential extraction unit. Revision to the ER is not proposed in response to this comment.

3. Comment: Additional qualitative/quantitative discussion addressing the degree of confidence applied to the radius of influence calculated at 9,500 feet for pilot well CPW1 would be useful for upcoming design and capture zone evaluations. (CPW1 Aquifer Parameter Test [Appendix E, page E-17]).

   Response: The radius of influence of 9,500 ft appears reasonable for a confined aquifer with transmissive characteristics in the range of that determined for the Lower Sand/Gravel Aquifer. A statement to that effect was incorporated into the text of Appendix E (see Section 3.2 in Attachment C).

4. Comment: A general planning question: Will the upper aquifer extraction system wells be completed at depth intervals inclusive of the Lacustrine Unit sand intervals, or are all completions planned for the Upper Sand/Gravel Unit?

   Response: It is anticipated that South System Phase II extraction wells will extend into Lacustrine Unit sand interbeds at some locations. Extension into the Lacustrine Aquitard sand interbeds will accomplish the dual purposes of intercepting contaminants that may have migrated into these sand interbeds, and providing additional available head for groundwater extraction.

MISCELLANEOUS LINE ITEM CORRECTIONS

1. Comment: Figure ER 2.11: "Pipe Diameter" should read "Trench Diameter".

   Response: Figure ER-2.11 was revised as suggested.

2. Comment: Section 3.5.3, Page 3-9, Third Paragraph, Second Line: Change "pilot" to "pitot" (?).

Response: Text was revised to indicate "pitot" instead of "pilot."

VERBAL COMMENTS PROVIDED DURING THE SEPTEMBER 26, 1991 MEETING

Verbal comments were provided by EPA and Ecology during a September 26, 1991 meeting between representatives of EPA, Ecology, Spokane County, E & E, and Landau Associates. The majority of the verbal comments were also provided in written form, and were previously responded to in this letter. However, two of the verbal comments were not incorporated into the written comments and are addressed herein.

1. Comment (EPA and Ecology): EPA and Ecology were expecting a greater portion of Phase II design to be incorporated into the Phase I Engineering Report. However, review of the Schedule for Submittal of Deliverables indicates the Phase I Engineering Report scope was in conformance with the agreed to scope. The Report should clarify the scope of the Phase I Engineering Report and the Phase II design submittals to be completed subsequent to the Phase I Engineering Report.

   Response: Sections 1.0 and 5.6 of the text and the Executive Summary, were revised to more clearly describe the scope of the ER, and the Phase II design process (see Section 5.6 in Attachment A)

2. Comment (EPA): The Executive Summary states that modifications proposed for the Phase II remedial systems (from that shown in the ROD conceptual design) can be made without revision to the ROD. This is a determination that can only be made by EPA. Please revise the text accordingly.

   Response: The text was revised to indicate that it appears that revision to the ROD is not required, but final determination as to whether the proposed Phase II remedial system modifications are in conformation with the ROD is the responsibility of EPA.

LANDAU ASSOCIATES, INC.

By:

Larry Beard

Lawrence D. Beard, P.E.
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cc: Washington State Department of Ecology (2 copies)
Spokane County (2 copies)

ATTACHMENT A

Revised Section 5.0

COLBERT LANDFILL
PHASE I ENGINEERING REPORT

5.0 CONCLUSIONS AND RECOMMENDATIONS

The objectives of Phase I of the Colbert Landfill RD/RA were to characterize hydrogeologic conditions and the extent of groundwater contamination in the vicinity of the proposed Phase II South, West, an East Interception/Extraction Systems, and to develop the parameters needed for design of the Phase II Remedial Action. These objectives were achieved. The data evaluations supporting achievement of these objectives are provided in Section 4.0 of this report and Appendices A through I. The extension of these data evaluations to conclusions and recommendations for Phase II design are presented in this section. Conclusions and recommendations are provided for the Phase II interception/extraction systems, Phase II groundwater monitoring, the Phase II treatment system, and the Phase II conveyance piping and outfall. Also provided is a summary of differences between the RI and Phase I interpretations of hydrogeologic conditions, and a description of the Phase II Remedial Design process.

5.1 HYDROGEOLOGIC CONDITIONS

Phase I has expanded the understanding of the hydrogeologic system in the Landfill vicinity from that developed in the RI. In some instances this increased knowledge of the hydrogeologic system will effect design of the final Remedial Action, and will result in differences between the conceptual design for the extraction systems provided in the ROD and the Phase II design. The differences between the hydrogeologic conditions described in the RI and conditions identified during Phase I are described in Sections 4.1 and 4.2 of this report. Significant differences in hydrogeologic conditions that are expected to impact Phase II design include:

- Groundwater flow (and constituent migration) in the Upper Sand/Gravel Aquifer diverts from a southerly direction toward the southeast near Woolard Road to a greater degree than identified in RI.
- A lobe of low permeability material (Latah Formation) that was not identified in the RI extends to the west into the Lower Sand/Gravel Aquifer beneath the Landfill. This lobe forms an east-west trending groundwater divide that separates the Lower Sand/Gravel Aquifer into northern and southern flow regimes in the Landfill vicinity.
- The horizontal hydraulic gradient in the Lower Sand/Gravel Aquifer west of the Landfill is significantly less than is estimated in the RI. As a result, groundwater flow (and contaminant migration) in the Lower Sand/Gravel Aquifer is slower than the RI estimate.

- The material properties and transmissive characteristics of the Weathered Latah Unit appear to be more similar to the Unweathered Latah Formation, than to the Basalt Unit (as characterized in the RI). As a result, the hydrogeologic units were redefined during Phase I to combined the weathered and unweathered Latah Formation (Unit D) and to separate out the Basalt Unit (Unit E) as an independent hydrogeologic unit.
- The transmissivity of Basalt Aquifer appears to be significantly lower than characterized in the RI. Also, no other hydrogeologic units were identified east of the Landfill that are sufficiently transmissive for effective groundwater extraction.

The impact that these changes in identified hydrogeologic conditions will have on design of the Phase II interception and extraction systems is described in Section 5.2.

5.2 INTERCEPTION/EXTRACTION SYSTEMS

The results of the Phase I hydrogeologic characterization (including Phase I APTs) indicate the Phase II South and West Interception Systems and the East Extraction System can be designed and implemented in accordance with the ROD and the SOW. However, conceptual design of the Phase II system indicates the locations and orientations of Phase II interception and extraction systems will vary significantly from the conceptual design presented in the ROD, as shown on Figure ER-5.1. It is important to recognize that the Phase II conceptual design shown on Figure ER-5.1 is only conceptual, and the number and location of extraction wells (and other system components) require significant additional design and analyses prior to finalization. interception/extraction system Phase II design recommendations that led to the Phase II conceptual design shown on Figure ER-5.1, and other recommendations developed during Phase I that will be incorporated into Phase II design, include:

- Phase II extraction wells and interception systems should be designed based on the aquifer parameters presented in Section 4.2 of this report. However, individual extraction wells should have an ultimate capacity about 20-25 percent greater than the design capacity to account for potentially variable aquifer conditions.
- The Phase II South Interception System should be located in the vicinity of Pilot Well CP-S1, and should be oriented in a northeast-southwest alignment (perpendicular to the apparent Upper Sand/Gravel Aquifer trough).
- The Phase II West Interception System should be oriented to address plume migration from the Northern and Southern Flow Regimes (shown on Figure ER-4.22).
- Because of the high transmissivity and relatively flat horizontal hydraulic gradients of the Lower Sand/Gravel Aquifer, the capture zones for Lower

Sand/Gravel Aquifer extraction wells are anticipated to be large. The large extraction well capture zones, combined with the slow advance of the plume in the Lower Sand/Gravel Aquifer, suggest the Phase II West Interception System should be located closer to the Landfill than indicated in the ROD conceptual model. These conditions may also make the distinction between the Phase II West Interception System and the East Extraction System (in the Lower Sand/Gravel Aquifer) unnecessary, because they will act as a single system.

- A sufficiently transmissive target aquifer (other than the Lower Sand/Gravel Aquifer) was not identified for the Phase II East Extraction System during the Phase I activities. The Basalt Aquifer is the only aquifer unit identified east of the Landfill, and its apparent capacity (about 5 gpm) limits the effectiveness of groundwater extraction in this area.

- Available data suggest that groundwater extraction east of the Landfill (particularly from the Basalt Aquifer) may induce contaminant migration toward the extraction well and spread contamination into areas to which it would not migrate under static (nonpumping) groundwater flow conditions (see Figure ER-4.23). It is recommended that Phase II East System groundwater extraction, for aquifers other than the Lower Sand/Gravel Aquifer, be limited to the immediate Landfill vicinity.

- Although anomalous constituent migration trends were observed east of the Landfill (during the RI and during Phase I), constituent migration appears to be reverting to trends more consistent with groundwater flow in areas where residences were connected to an alternative water supply and are no longer using private wells. It is recommended that additional residences near the plume boundary east and northeast of the Landfill be connected to alternative water, and private well pumping in these areas cease for a sufficient period of time (years) to determine if groundwater extraction is the cause of anomalous constituent migration in these areas. Recommendations for alternative water hookups will be provided during preliminary Phase II design.

- Well performance difficulties experienced during the CP-E1 APT indicate that certain design, operation, and maintenance features should be applied to Phase II extraction wells. These include:

  - Entrance velocities should be lower than commonly applied design values to minimize the potential for chemical scale formation

  - Pumping systems should use epoxy and plastic coated well discharge pipe instead of galvanized steel pipe

  - Extraction wells should be treated for biological growth following initial construction

  - Extraction wells should be inspected via downhole camera periodically (about every 3-6 months, initially) during Phase II for chemical and/or biological scale formation, and treated (as needed) to maintain efficient well performance.

5.3 GROUNDWATER MONITORING

Groundwater monitoring is required downgradient of the South and West Interception Systems to evaluate system performance. The SOW specifies three to eight downgradient monitoring wells for the South System, and six downgradient monitoring well locations for the West System. The SOW also gives Spokane County the option of installing (at its discretion) up to three additional monitoring wells to better characterize hydrogeologic conditions and contaminant distribution in the Upper Sand/Gravel Aquifer. Phase II groundwater monitoring recommendations include:

- Downgradient groundwater monitoring for the South Interception System should include three to eight new monitoring wells. The monitoring wells should be installed downgradient of the zone of capture, which will be estimated during Phase II design.
- Based on Phase I data, Monitoring Well Locations CD-40 through CD-45 appear to be appropriately located for West Interception System downgradient groundwater monitoring. However, Phase II design (capture zone analysis) should be performed prior to final determination of West System groundwater monitoring locations.
- One to two additional monitoring wells should be installed in the Upper Sand/Gravel Aquifer to evaluate the potential for constituent migration to the south (in addition to the southeast) near Woolard Road. The monitoring well(s) should be located between US Highway 2 and Yale Road, about 500-1,000 ft south of Woolard Road.

5.4 TREATMENT SYSTEM

Treatability Study results indicate air stripping can achieve the removal efficiencies necessary to meet the Performance Standards for Phase II effluent discharge, and methylene chloride will be the Constituent of Concern controlling Phase II treatment system design. The air stripping performance data obtained from the Treatability Study will be used to calibrate and optimize the Phase II treatment system design. Chemical analyses and visual observation during Phase I indicate chemical scale control will be needed for the Phase II treatment system. The Phase I air emissions abatement assessment indicates that Phase II air stripping tower air emissions abatement will not be needed. Phase II treatment system design recommendations include:

- Extracted groundwater from the Phase II South, West, and East Interception/ Extraction Systems should be treated at a single facility located near the southwest corner of the Landfill (see Figure ER-5.1 for location).

- The method of chemical scale control should be selected based on cost effectiveness and discharge considerations.
- The Phase II air stripping tower should be designed to meet the Performance Standards, based on hydraulic loadings and constituent concentrations estimated from design of the Phase II interception/extraction systems.
- The need for additional health risk assessment of air stripping tower air emissions should be evaluated following preliminary design of the Phase II treatment system. If the preliminary Phase II design includes system components or operating conditions likely to result in higher airborne emissions than those predicted for the Phase I assessment, air quality modeling should be performed using the Phase II treatment system design parameters for model emission source input.
- Based on the Phase I air emissions abatement assessment, detailed air quality modeling using onsite meteorological data does not appear warranted. However, onsite meteorological data should be collected for at least one year (through 1991) so adequate onsite data are available, if needed.

5.5 PIPING AND DISCHARGE SYSTEMS

Piping will be needed to convey groundwater from extraction wells to the treatment facility, and from the treatment facility to the outfall. The conveyance piping constructed during Phase I was designed for anticipated Phase II flows, and is appropriate for use during Phase II. Phase II piping and discharge system(s) recommendations include:

- Additional Phase II conveyance piping should be constructed (as needed) to supplement the existing piping system, using specifications similar to those used for Phase I.
- The existing (Phase I) outfall to the Little Spokane River should be used for Phase II discharge (see Figure ER-5.1 for location). However, recharge of treated water to the subsurface is contained in the SOW as a discharge option, and should be retained for potential use during remedial action. The Phase I South Infiltration System will not be used for Phase II Remedial Action.

5.6 PHASE II DESIGN

5.6.1 Regulatory Agency Concurrence

The Schedule for Submittal of Deliverables (Landau Associates 1989f) specifies that preliminary Phase II work plans, which are the initial Phase II design submittals (as described in Section 5.6.2), are to be submitted 105 days following EPA and Ecology approval of this report. It is recognized that EPA and Ecology approval of all of the characterizations, analyses, conclusions, and recommendations contained in this report may not be practicable because of the interpretive and subjective nature of some of the analyses and conclusions. However, EPA and Ecology concurrence with the following key conclusions are critical to timely and cost-effective design of the Phase II Remedial Action:

- The aquifer transmissivity values presented in Section 4.2 of this report are appropriate for Phase II design.
- Air stripping is capable of achieving the Performance Standards for extracted groundwater.
- The preliminary air emissions abatement assessment presented in Section 4.5 is adequate and offgas treatment will not be required for the Phase II stripping tower, provided the additional evaluation to be performed during Phase II design (as described in Sections 4.5.3.3 and 5.4 of this report) support the preliminary assessment.

Phase II design will be initiated following EPA and Ecology concurrence with these key conclusions and approval to proceed with Phase II design.

5.6.2 Phase II Design Submittals and Schedule

Phase II design submittals are described in the Schedule for Submittal of Deliverables (Landau Associates 1989f). Thirty percent design will be incorporated into preliminary Phase II Groundwater Monitoring, Extraction Well, and Treatment and Discharge work plans. Sixty percent design will incorporate EPA and Ecology comments on the preliminary work plans, and will be contained within the final work plans. Preparation of plans and specification will be initiated subsequent to EPA and Ecology review and approval of the final Phase II work Plans. Preliminary Plans and Specifications will constituent the 90 percent design submittal. Final Plans and Specifications will be prepared subsequent to EPA and Ecology review and comment on the preliminary submittal, and will constitute 100 percent (final) design.

The Project Health and Safety Plan and QAPjP will be updated early in the design process and submitted for EPA and Ecology review and approval. The Project Operation and Maintenance Plan will be prepared, and submitted to EPA and Ecology for review, concurrently with the (preliminary and final) Plans and Specifications.

The Phase II design process is anticipated to require about 15 months. However, several EPA and Ecology design reviews are incorporated into Phase II design, and the actual time required for design will be dependent on good communication between designers and reviewers, and timely submittal of design review comments. At present, one County-Agency technical session is scheduled for the 50 percent completion stage of preliminary Phase II work plan preparation (Landau Associates 1989f). Additional County-Agency technical sessions may be appropriate at other stages of Phase II design to maintain adequate communication, and may be requested by the County (or EPA or Ecology) during Phase II design. The Phase II design schedule is provided on Figure ER-5.2.

5.6.3 Bases for Design

The SOW provides the bases for design of the Phase II Remedial Action for groundwater interception, treatment, and monitoring. The Phase II Remedial Action will be designed to conform to the SOW, utilizing data and analyses developed during Phase I.

The Phase II South and West Interception Systems and East Extraction System will be designed utilizing the aquifer parameters, boundary conditions, and constituent distribution presented in this report. Due to the complexity of aquifer boundary conditions, a numerical groundwater flow model will be utilized for interception/extraction system capture zone analyses, and assessment of the impact of the Remedial Action on private wells. The Phase II downgradient groundwater monitoring system will be designed such that monitoring wells are outside of the zone of capture of the Phase II interception/extraction systems. Groundwater flow model documentation and results will be presented in the preliminary and final Phase II Extraction Well Plan for EPA and Ecology review.

The Phase II treatment system will be designed utilizing the results of the Phase I pilot studies, groundwater modeling results, and published vendor information. The hydraulic loadings for treatment system design will be based on groundwater flow model results. Treatment system influent concentrations for the Constituents of Concern will be estimated using a solute transport model, coupled with the groundwater flow model used for interception system design. The solute transport model will also be used to estimate influent alkalinity concentrations, which are needed for developing Phase II chemical scale control requirements. Solute transport modeling results will be presented in the Phase II Treatment and Discharge Plan for EPA and Ecology review.

The operating and performance data developed during the Treatability Study, as well as the estimated influent concentration and effluent water quality criteria, will be used to develop baseline performance criteria for the air stripping system. This system performance criteria will be used to prepare a performance specification for the Phase II treatment system (which will be used for system procurement), and will be used to compare manufacturer-proposed equipment to the performance of the pilot Treatment System. The acceptability of the manufacturer-proposed Phase II air stripping system will be evaluated using the model equations and performance data from the Treatability Study.

The performance specification will include treatment system performance requirements, pretreatment/posttreatment requirements, and process instrumentation and control requirements for procurement of the air stripping system. The pilot treatability study results will be provided as part of the specification, and will also be used to evaluate the adequacy of the manufacturer-proposed air stripping system. Shop drawings for the air stripping system will be prepared by the manufacturer. Plans and specifications and shop drawings for other aspects of the treatment facility (piping, pumps, tanks, instrumentation and control, and buildings) will be prepared by Landau Associates or its subconsultants.

Much of the Phase II piping and discharge system was constructed during Phase I (see Section 2.3). Consequently, Phase II design for conveyance piping and discharge systems will consist primarily of designing the piping system to convey extracted water from the South System to the central treatment facility and piping from individual extraction wells to existing pipelines. Phase II pipelines will be designed based on the hydraulic loadings developed from groundwater modeling results. Phase II discharge will be to the Little Spokane River using the existing pipeline and outfall. The additional hydraulic loading to the Little Spokane River resulting from the Phase II discharges is not anticipated to significantly impact downstream flows. However, the impact of the outfall on river flows will be assessed during Phase II design.

5.6.4 Phase II Permitting

Section XXI of the Consent Decree specifies that no federal, state, or local permit shall be required for the portions of the Remedial Action conducted entirely on the Site, although compliance with the substantive requirements of all applicable federal laws is required. The Consent Decree also specifies (Section XXI) that the Remedial Action is exempt from the procedural and substantive requirements of state and local laws. Because all remedial action will occur within the Site boundary (as shown on Figure ER-1.1), permitting issues are limited to substantive compliance with federal laws. The ROD identifies the following federal laws and regulations as applicable, or relevant and appropriate requirements (ARARs):

- Resource Conservation and Recovery Act (RCRA) (42 USC 6901), Subtitle C:
  - Protection of groundwater (40 CFR 264, Subpart F)
  - Closure and post-closure of landfills (40 CFR 264, Subpart G)

- Safe Drinking Water Act (SDWA) (42 USC 300):
  - Drinking Water Standards (40 CFR 141), including both enforceable maximum contaminant levels (MCLs) and recommended maximum contaminant levels (RMCLs).
- Clean Water Act (CWA) (33 USC 1251):
  - National Pollutant Discharge Elimination System (NPDES) (40 CFR 122)
- Clean Air Act (CAA) (72 USC 7401):
  - National Emission Standards for Hazardous Air Pollutants.

Substantive compliance with the Clean Water Act National Pollutant Discharge Elimination System (NPDES) program will require submittal of anticipated effluent discharge information and compliance with surface water monitoring requirements. Discharge information will include anticipated hydraulic loadings and constituent concentrations (including both the Constituents of Concern and other appropriate constituents). Surface water monitoring requirements are anticipated to include establishment of monitoring parameters, sampling location(s), and sampling and reporting frequency. Anticipated discharges and proposed sampling and reporting to address NPDES substantive compliance will be addressed in the Preliminary (30 percent design) and Final (60 percent design) Phase II Treatment and Discharge Plan.

Substantive compliance with the Clean Air Act National Emission Standards for Hazardous Air Pollutants will require assessment of the need for application of best available control technology (BACT) to stripping tower air emissions and compliance with air monitoring and reporting requirements. The basis for determining the need for BACT is provided in Section V.D. of the SOW, and the preliminary assessment is provided in Section 4.5 of this report. Final assessment of the need for BACT will be provided in the Final Phase II Treatment and Discharge Plan. Proposed sampling and reporting of Phase II air emissions will be presented in the Preliminary and Final Phase II Treatment and Discharge Plan.

Substantive compliance of the Remedial Action with other identified ARARs (i.e., RCRA and SDWA) are not anticipated to necessitate submittals or monitoring beyond that specified in the SOW.

* * * * *

This Phase I Engineering Report was prepared for Spokane County to fulfill the requirements of the Consent Decree for the Colbert Landfill Remedial Design/Remedial Action. This report is intended to provide the U.S. Environmental Protection Agency and the Washington State Department of Ecology with a general understanding of Phase I activities and evaluation results, and how Phase I results will affect the design of the Phase II (final) Remedial Action for the Colbert Landfill Project.

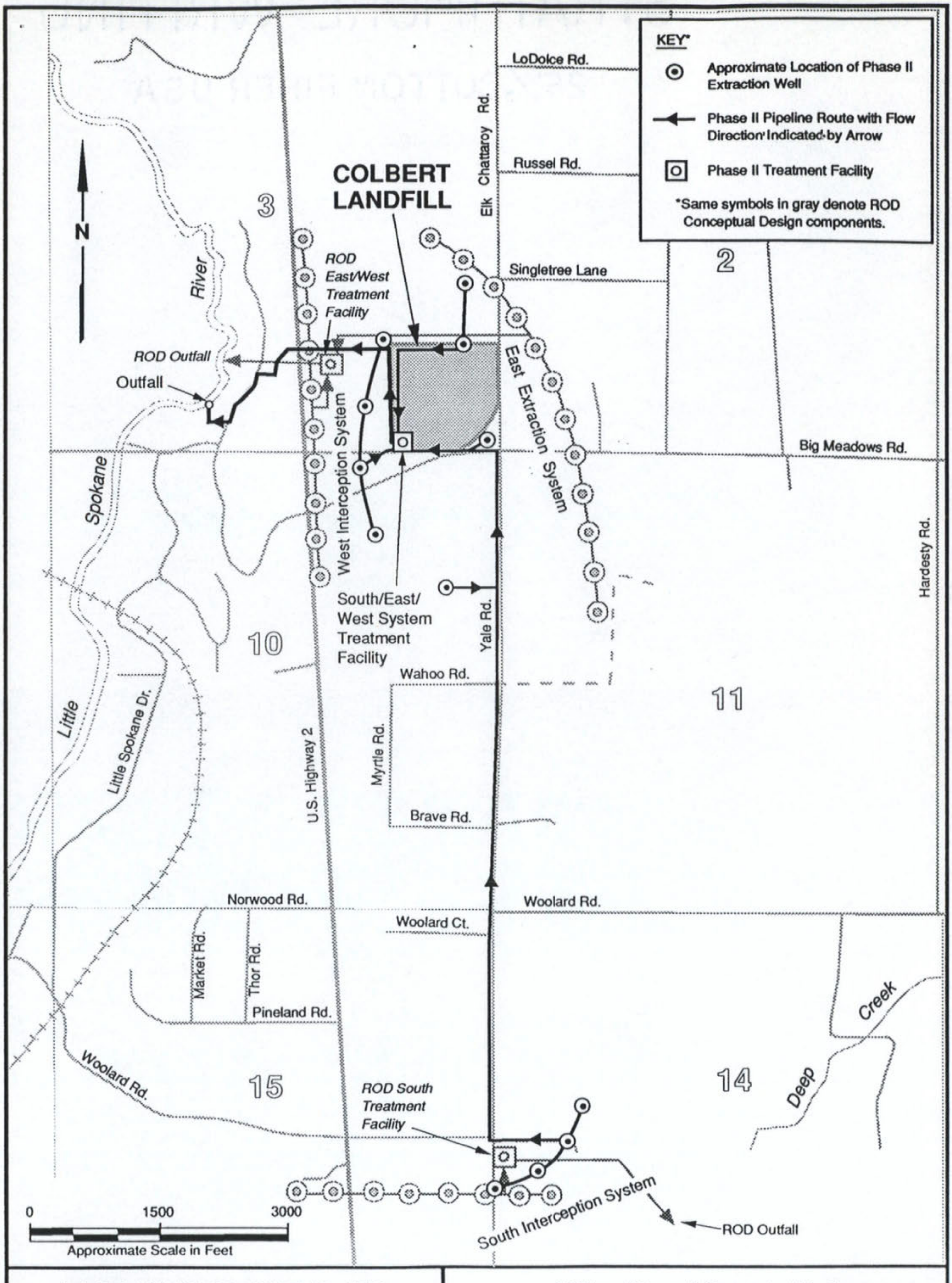
Phase I activities were conducted in accordance with generally accepted engineering practices at the time these activities were accomplished, and in general accordance with the Phase I work plans which incorporated U.S. Environmental Protection Agency and Washington State Department of Ecology comments. No other warranty or representation, express or implied, is applicable.

LANDAU ASSOCIATES, INC.

By:

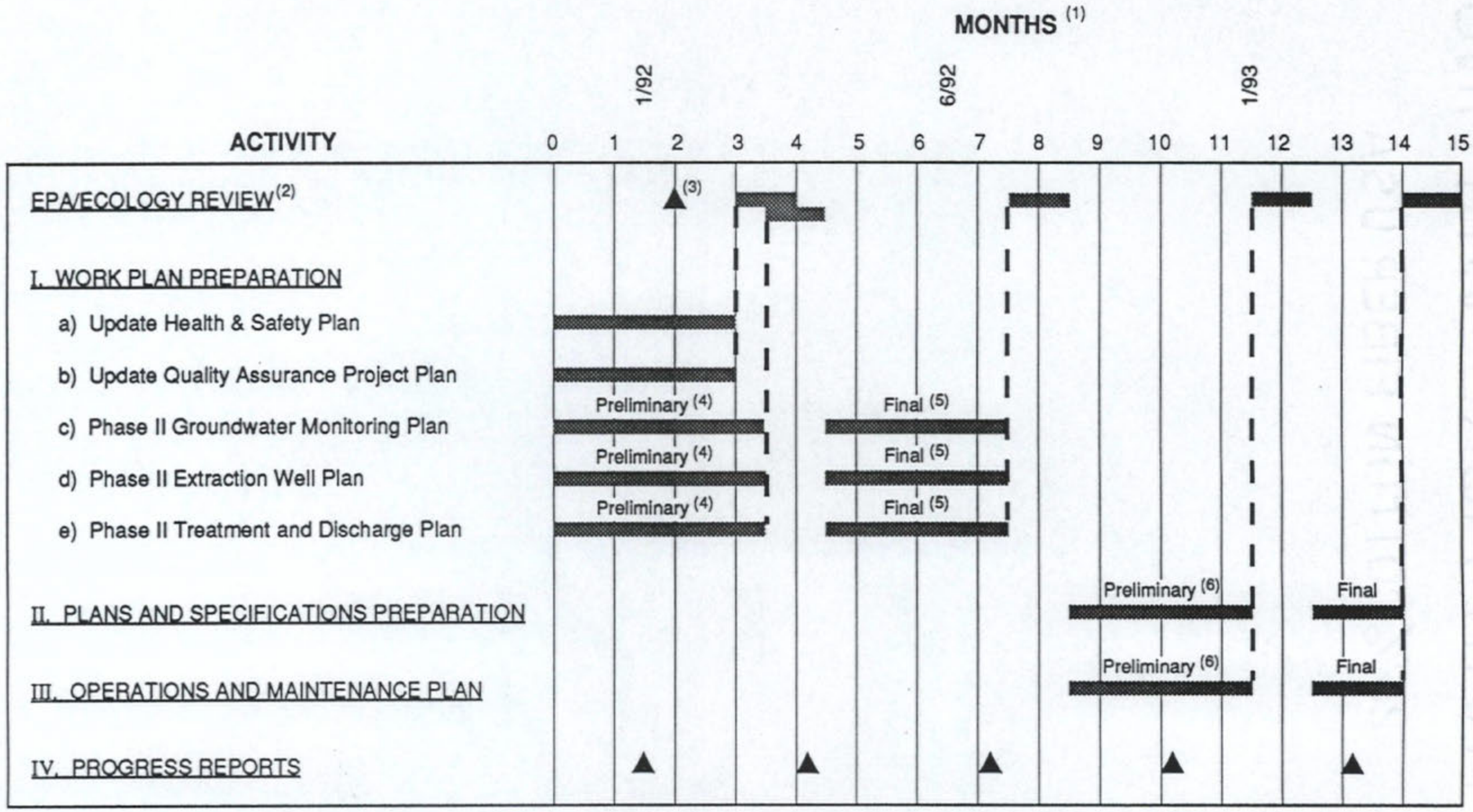
Lawrence D. Beard, P.E.
Project Manager

LDB/njb
No. 124-01.61



LANDAU ASSOCIATES, INC.	ROD and Phase II Conceptual Design

Figure ER-5.1



NOTES

1. 1/92 ← Date

1 2 ← Time in months since EPA/Ecology approval to proceed with Phase II design.

2. Estimated Schedule. Schedule dependent on actual EPA/Ecology review period.
3. County-Agency technical session at 50% completion of preliminary work plans.
4. Represents 30% Design Submittal.
5. Represents 60% Design Submittal.
6. Represents 90% Design Submittal.

Source: Schedule for Submittal of Deliverables (Landau Associates, 1989).

LANDAU ASSOCIATES, INC.

Phase II Design
Estimated Schedule

Figure ER-5.2

ATTACHMENT B

Revised Sections 4.2, 4.3, 4.4

COLBERT LANDFILL

PHASE I ENGINEERING REPORT

4.2 HYDROGEOLOGY

The Phase I characterization of Site hydrogeologic conditions is presented in this section. This characterization is based on subsurface geologic information collected during Phase I and previous investigations, groundwater elevation data collected during Phase I, and Phase I APT results. As stated in the SOW, the primary purpose of this Phase I hydrogeologic characterization is to develop the aquifer parameters (such as transmissivity and hydraulic conductivity) needed for design of the Phase II extraction, treatment, and discharge system(s). Hydrogeologic characterization is also needed to characterize contaminant migration for proper Phase II extraction well placement, and for estimation of Phase II contaminant mass loading to the treatment system.

This hydrogeologic characterization expands and refines the hydrogeologic model developed in the RI. Where appropriate, comparisons are made between hydrogeologic properties presented in the RI and those developed in this report. These comparisons indicate how differences (where they exist) between the RI and the Phase I assessment may impact Phase II design.

4.2.1 Aquifer Design Parameters

The primary aquifer parameters required for Phase II design are transmissivity (T) and hydraulic conductivity (K). Also of value for short-term (transient) aquifer response is the storage coefficient; storativity (S) for confined aquifer conditions, and specific yield (Sy) for unconfined aquifer conditions.

These parameters were estimated based on analysis of the Phase I APT data. A number of analyses were utilized for data evaluation. In general, analyses included a semilogarithmic straight line analysis of drawdown and recovery data versus time, semilogarithmic straight line analysis of drawdown versus distance, and curve matching analyses of drawdown versus time data plotted on a log-log scale.

Curve matching techniques allow consideration of boundary conditions (such as leaky aquifers) and delayed yield (for unconfined aquifers), and typically provide the most accurate assessment of aquifer parameters. Semilogarithmic analyses were utilized, although these straight line analyses do not account for leaky aquifer conditions or delayed yield (prevalent conditions for the Site) and tend to overestimate transmissivity when these conditions are present.

The aquifer parameters presented in this section include reasonable upper and lower bound estimates of transmissivity and average estimated values for the storage coefficient. Because the average transmissivity values include values derived from straight line analyses that tend to overestimate transmissivity when applied to conditions prevalent for the site, they are presented as reasonable upper bound values in this report. The lower bound transmissivity values presented in this report are based on judgment and experience, with consideration given to the analyses applied and resulting transmissivity estimates. Additional discussion of APT data evaluation, including supporting data and analyses, are presented in Appendix E.

Upper bound transmissivity values will be used during Phase II design for estimating maximum hydraulic loading, individual well design (sizing), and pipeline design. Lower bound transmissivity values will be utilized for estimating minimum hydraulic loading, and extraction well spacing and/or available head for drawdown.

4.2.2 Hydrogeologic System

The hydrogeologic system in the Landfill vicinity can be characterized as containing four aquifers (two primary and two secondary) and three aquitards:

- The Upper Sand/Gravel Unit (Unit A) forms the Upper Sand/Gravel Aquifer when underlain by the Lacustrine Unit (Unit B), and is considered a primary aquifer.
- The Lacustrine Unit (Unit B) is the low-permeability unit that separates the Upper and Lower Sand/Gravel Units and is referred to as the Lacustrine Aquitard. The Lacustrine Aquitard does contain water-bearing sand layers and, based on water elevation data, some of the shallow sand layers appear to be hydraulically connected to the Upper Sand/Gravel Aquifer.
- The Lower Sand/Gravel Unit (Unit C) forms the Lower Sand/Gravel Aquifer, which is the second primary aquifer (and the regional aquifer for the Site).
- The Latah Formation (Unit D), and the Weathered Latah Subunit (Unit D_1), serve as the aquitard underlying the Lower Sand/Gravel Aquifer at most locations and (in combination) are referred to as the Latah Aquitard. However, some low-yield private wells are installed in the Latah Aquitard to the east of the Landfill, where the Upper and Lower Sand/Gravel Aquifers are not present.
- The Basalt Unit (Unit E) forms a secondary aquifer interbedded with the (combined) Latah Aquitard, and is referred to as the Basalt Aquifer.
- The Granite Unit (Unit F) serves as the lower boundary (aquitard) to the regional flow system, although some low-productivity wells are installed in the upper portion of this unit.

The Fluvial Unit associated with the Little Spokane River forms the Fluvial (secondary) Aquifer. The Fluvial Aquifer may be in direct hydraulic connection with the Lower Sand/Gravel Aquifer, but piezometric and contaminant migration data (as discussed in subsequent sections of this report) suggest that it be treated as an independent hydrogeologic unit for the purposes of this Project.

The hydrogeologic units described above are in general agreement with those presented in the RI, with two exceptions. The RI combined the Weathered Latah Subunit and Basalt Unit and treated them as a single aquifer, and characterized the (unweathered) Latah Formation as a separate aquitard. Although the Basalt Unit does act as a secondary aquifer (described herein as the Basalt Aquifer), the Weathered Latah Subunit does not appear to be sufficiently more transmissive than the underlying Latah Formation to warrant characterization as an aquifer. Consequently, the weathered and unweathered portions of the Latah Formation (exclusive of the Basalt Unit) are collectively referred to as the Latah Aquitard (Unit D), and the Basalt Unit is independently referred to as the Basalt Aquifer (Unit E).

Phase I hydrogeologic characterization activities are focused towards development of hydrogeologic design parameters and groundwater flow characteristics in the aquifers identified for groundwater extraction as part of the South, West, and East Phase II Interception/Extraction Systems. Consequently, the Upper Sand/Gravel Aquifer (South System), Lower Sand/Gravel Aquifer (East and West Systems), and the Basalt Aquifer (East System) were the focus of most Phase I hydrogeologic investigation and evaluation activities. The following subsections address these aquifer units. The characteristics of aquitards and other aquifer units are discussed (where appropriate) in the context of these units. However, a separate discussion of the hydrogeologic characteristics of the Fluvial Aquifer, and the Lacustrine, Latah, and Granite Aquitards is not presented in this report.

4.2.3 Upper Sand/Gravel Aquifer

4.2.3.1 Nature and Extent

The Upper Sand/Gravel Aquifer is unconfined with the water table about 90 ft below ground surface (BGS). The saturated thickness varies from less than 1 ft to greater than 19 ft, as shown on Figure ER-4.14. The aquifer appears to thicken towards the center of a northwest-southeast trending trough that extends from west of the Landfill to, and probably southeast of, Monitoring Well CD-30A. This apparent trough may be a buried stream channel or an erosional feature created during the Glacial Lake Missoula floods.

Due to the erratic nature of the upper (erosional) surface of the Lacustrine Aquitard and the presence of sand interbeds within the Aquitard, the distinction between the base of the Upper Sand/Gravel Aquifer and shallow sand interbeds within the Lacustrine Aquitard is often difficult to discern. Groundwater elevation data from Monitoring Well CD-32B1 (screened in shallow sand interbeds of the Lacustrine Aquitard) suggest a direct hydraulic connection between the Upper Sand/Gravel Aquifer and shallow Lacustrine Aquitard sand interbeds, at least in the vicinity of the apparent trough. It is probable that some of the "shallow" private wells are screened in the Lacustrine Aquitard sand interbeds rather than the Upper Sand/Gravel Aquifer.

Groundwater flow in the Upper Sand/Gravel Aquifer is shown on Figure ER-4.15 and is generally from north to south, deflecting to the southeast about 1 mile south of the Landfill. Based on groundwater elevation data and contaminant migration data (Section 4.3), groundwater appears to be influenced by the topography of the upper surface of the Lacustrine Aquitard and flows along (and within) the apparent trough. The general trends in groundwater flow are consistent with those presented in the RI.

The areal extent of the Upper Sand/Gravel Aquifer is controlled by the presence (or absence) of the Lacustrine Aquitard. The Upper Sand/Gravel Aquifer is truncated along its western margin by the Little Spokane River Valley and on its east margin by the discontinuation of the Lacustrine Aquitard. These lateral boundaries (for the extent to which they are known or can be inferred) are shown on Figure ER-4.15. The north and south boundaries for the Upper Sand/Gravel Aquifer are outside the Phase I investigation area, and remain undefined.

The only identified source of significant recharge to the Upper Sand/Gravel Aquifer is the direct infiltration of precipitation. Recharge may also occur from the north and southeast, although only limited data are available in these areas. Long-term water elevation fluctuations in the Upper Sand/Gravel Aquifer are relatively minor, as shown on Figure ER-4.16 and Table ER-3.1. This lack of seasonal water level fluctuation suggests a significant recharge source other than direct infiltration of precipitation, although this significant recharge source has not been identified.

Discharge from the Upper Sand/Gravel Aquifer occurs primarily to the south or southeast of the Phase I investigation area, although the ultimate discharge location is undefined. Discharge from the Upper Sand/Gravel Aquifer also occurs along its western margin, as evidenced by the springs along the bluff overlooking the Little Spokane River. Although these springs do not appear to be a major discharge boundary for the aquifer, they are significant because they provide a migration path for the Constituents of Concern to the Fluvial Aquifer near the Little Spokane River. Additional discharge occurs to the east in the immediate Landfill vicinity, where the Lacustrine Aquitard pinches out. Although the rate of discharge to the east may not be significant, it has a significant impact on contaminant distribution in the underlying aquifers (as discussed in Section 4.2.6).

4.2.3.2 Aquifer Parameters

Aquifer parameters for the Upper Sand/Gravel Aquifer are based on analyses of CP-S1 APT data, and are summarized in Table ER-4.1. A description of the data analyses, along with data plots and sample calculations, are provided in Appendix E. Analysis of CP-S1 APT data indicate upper and lower bound estimates for T of about 12,000 ft^2/day and 10,000 ft^2/day, respectively. K is estimated to range between about 640 and 530 ft/day, based on the range in estimated T values and an estimated saturated thickness of 19 ft. Sy is estimated to be about 0.20. Distance-drawdown data indicate that the radius of influence of Pilot Well CP-S1 for a discharge rate of 95 gpm is about 1,000 ft. Based on these estimated hydraulic conductivities, horizontal hydraulic gradients estimated from Figure ER-4.15, and an assumed effective porosity of 0.3, the average linear velocity for groundwater in the Upper Sand/Gravel Aquifer is estimated to range between about 3.5 and 6.4 ft/day, as shown in Table ER-4.2.

The estimated T is higher than the estimates provided in the RI (summarized in Table ER-4.1), although the hydraulic conductivity and velocity estimates compare well. The difference in T appears to be primarily the result of a difference in estimated saturated thickness (8 ft in the RI versus 19 ft for Phase I).

Pilot Well CP-S1 well efficiency was estimated to be about 64 percent using the method described in Appendix E. This method estimates well efficiency based on the ratio of observed to theoretical specific capacity for the pumping well. This estimated well efficiency and the effects of well partial penetration and aquifer dewatering will be considered during design of the Phase II South Interception System.

Step drawdown tests may provide a more accurate means of assessing well efficiency, and are commonly used to assess the maximum pumping rate at which a well maintains a high level of efficiency. However, step drawdown tests could not be performed during Phase I due to the limitations in discharge rate adjustment imposed by the Treatment System. It is anticipated that performing step drawdown tests will be possible during Phase II. However, Phase II extraction wells will be designed using lower than normal groundwater entrance velocities due to the continuous, long-term nature of pumping, and determining maximum pumping rates for efficient well operation is not expected to be significant consideration for Phase II design or operation. Of primary concern for Phase II operation will be assessing changes in well efficiency over time, which can be accomplished by evaluating trends in specific capacity (ratio of discharge rate to drawdown in the pumping well). Consequently, step drawdown tests will probably not be accomplished for Phase II extraction wells.

Pilot Well CP-S1 is screened within the apparent trough of thicker (and probably coarser) material in the Upper Sand/Gravel Aquifer. It is anticipated that T and K for Phase II extraction wells installed towards the lateral limits of the trough will be lower than those obtained from the CP-S1 APT.

4.2.4 Lower Sand/Gravel Aquifer

4.2.4.1 Nature and Extent

The Lower Sand/Gravel Aquifer is a confined system to the west of the Landfill and is unconfined (or semiconfined) east of the western Landfill boundary. The aquifer potentiometric surface is about 180 ft BGS and the saturated thickness varies from 0 east of the Landfill to over 220 ft near U.S. Highway 2, as shown on Figure ER-4.17. The saturated thickness varies from 0 to greater than 120 ft beneath the Landfill. The thickest portion of the aquifer may define a north-south trending trough. However, this trough does not appear to significantly influence the direction of groundwater flow. The impact of the lobe of Latah Aquitard that extends to the west into the Lower Sand/Gravel Aquifer in the Landfill vicinity is evident from the deflection of the zero-feet saturated thickness contour to the west, and the rapid increase in saturated thickness extending radially out from that contour.

To the west of the Landfill, the Lacustrine Aquitard separates the Upper and Lower Sand/Gravel Aquifers and is the overlying confining layer for the Lower Sand/Gravel Aquifer. The Lower Sand/Gravel Aquifer is directly underlain by the Latah Aquitard and, in places, by the Granite Aquitard. The eastern boundary of the Lower Sand/Gravel Aquifer is defined by the Latah Aquitard as it rises to the east and extends above the Lower Sand/Gravel Aquifer groundwater surface (represented on Figure ER-4.17 as the zero saturated thickness contour). The northern, western, and southern boundaries for the aquifer are outside the Phase I investigation area. However, the strong upward vertical gradient near the Little Spokane River, observed at Monitoring Well Location CD-40 (see Figure ER-4.18), indicates the river provides a partial western hydraulic boundary for the Lower Sand/Gravel Aquifer.

Groundwater elevation data collected for the Lower Sand/Gravel Aquifer during Phase I were often collected in conjunction with groundwater elevation measurements for wells in the Latah, Basalt, and Granite Units to the east of the aquifer because these units are hydraulically connected, to varying degrees. Collectively, these units are referred to as the Lower Aquifers. Groundwater flow in the Lower Sand/Gravel Aquifer and the combined Lower Aquifers are shown on Figures ER-4.19 and ER-4.20, respectively.

The Landfill is located in an area of converging groundwater flow for the Lower Sand/Gravel Aquifer. Groundwater flow is generally to the west. However, groundwater flow north of the Landfill is to the west-southwest, and groundwater flow south of the Landfill is to the northwest. Additionally, the horizontal hydraulic gradients for the Lower Sand/Gravel Aquifer decrease significantly to the west of the Landfill as the aquifer thickens.

The lobe of Latah Aquitard that extends into the aquifer from the east appears to separate the Lower Sand/Gravel Aquifer into two flow regimes in the Landfill vicinity. A Southern Flow Regime is present to the south of the Landfill, and the Deep Creek drainage appears to be its primary recharge source. A Northern Flow Regime is present to the north of the Landfill, and the Deer Creek drainage (located about 1.5 miles north of the Landfill) may be its primary source of recharge. However, groundwater elevation data north of the Landfill is limited and the recharge source for the Northern Flow Regime is not well defined. The Northern and Southern Flow Regimes appear to converge to the west of the Landfill.

The Weathered Latah Subunit and Basalt Aquifer to the east of the Landfill was characterized as the primary recharge source for the Lower Sand/Gravel Aquifer in the RI. However, the apparent lack of impact of the area immediately east of the Landfill on groundwater flow direction, as shown on Figures ER-4.19 and ER-4.20, suggests that the hydrogeologic units immediately east of the Landfill do not provide a significant percentage of the recharge to the Lower Sand/Gravel Aquifer.

Long-term water level elevations for the Lower Sand/Gravel Aquifer, collected from Monitoring Well CD-3L, are presented on Figure ER-4.16, and indicate an annual water level fluctuation of about 1 ft. Maximum and minimum water levels for Monitoring Well CD-3L correspond to common seasonal variations of higher water elevations in spring to early summer and lower water elevations in late summer to fall.

The Little Spokane River appears to be the primary discharge location for the Lower Sand/Gravel Aquifer. This conclusion is supported by the upward hydraulic gradient at Monitoring Well Location CD-40 shown on Figure ER-4.18, and the relationship between Little

Spokane River stage elevation and groundwater elevations for Monitoring Well Location CD-40 (shown on Figure ER-4.21). The relatively steep upward gradient between Monitoring Wells CD-40C1 and CD-40C2 could also be, in part, the result of hydrogeologic separation between the wells; if so, Monitoring Well CD-40C1 would be characterized as screened in the Fluvial Aquifer and CD-40C2 screened in the Lower Sand/Gravel Aquifer.

4.2.4.2 Aquifer Parameters

Aquifer parameters for the Lower Sand/Gravel Aquifer are based on results from APTs conducted for Pilot Wells CP-W1 (West System) and CP-E1 (East System), and are summarized in Table ER-4.1. A description of the data analyses, along with data plots and calculations for these APTs, are provided in Appendix E.

Data analyses indicate T and K in the vicinity of Pilot Well CP-W1 are between about 30,000 to 40,000 ft^2/day and 170 to 230 ft/day, respectively. T and K in the vicinity of Pilot Well CP-E1 are estimated to be about 10,000 to 14,000 ft^2/day and 100 to 140 ft/day, respectively. The higher K values to the west (at Pilot Well CP-W1) may result from coarser materials having been deposited towards the center of the Lower Sand/Gravel Unit (to the west), although a clear trend of coarser materials to the west was not observed in boring samples. At Pilot Well CP-W1 (where the aquifer is confined) S is estimated to be 0.0004. At Pilot Well CP-E1 (where the aquifer is unconfined) Sy is estimated to be about 0.16. Based on estimated hydraulic conductivities, horizontal hydraulic gradients estimated from Figure ER-4.19, and an assumed effective porosity of 0.3, average linear velocity is estimated to range from 0.3 ft/day (near Pilot Well CP-E1) to 0.6 ft/day (near Pilot Well CP-W1), as shown in Table ER-4.2.

APT distance-drawdown data indicate the radius of influence for Pilot Well CP-W1, operated at a pumping rate of 220 gpm, is about 9,500 ft and the radius of influence for Pilot Well CP-E1, operated at a pumping rate of 200 gpm, is about 3,500 ft. The radius of influence of both of these wells extends beyond the eastern aquifer boundary. Consequently, long-term pumping during Phase II is anticipated to result in discharge boundary effects for East System and West System Phase II extraction wells installed in the Lower Sand/Gravel Aquifer.

The Basalt Aquifer responded to Lower Sand/Gravel APTs at some locations (CD-4, CD-8), but not other locations (CD-7, CD-20), indicating a direct (although incomplete) hydraulic connection between these aquifers. No response in Upper Sand/Gravel Aquifer water levels was observed during Lower Sand/Gravel Aquifer APTs.

Well efficiencies were estimated at about 78 and 85 percent for Pilot Wells CP-E1 and CP-W1, respectively, using the method described in Appendix E. These well efficiencies, and the effects of partial penetration, will be considered during II design of the West Interception System and East Extraction System.

Vertical hydraulic gradients for the Lower Sand/Gravel Aquifer were evaluated for monitoring well locations CD-21, CD-40, CD-41, CD-42, and CD-43, and are shown on Figure ER-4.18. These data indicate a general upward gradient throughout the Lower Sand/Gravel Aquifer, with the upward vertical gradient increasing towards the Little Spokane River.

The estimated values for T and K compare well with those presented in the RI (as shown in Table ER-4.1). However, the RI estimated the average linear velocity for the Lower Sand/Gravel Aquifer to be between 2 and 12 ft/day, much greater than that estimated during Phase I. The higher velocity estimate in the RI appears to be the result of an overestimation of horizontal hydraulic gradient to the north and west at the Landfill, and is probably due to the limited data available for this area during the RI.

4.2.5 Basalt Aquifer

4.2.5.1 Nature and Extent

One objective of the Phase I program was to identify transmissive units for groundwater extraction in the Lower Aquifers east of the Colbert Landfill. Of the three initial Phase I monitoring well locations east of the Landfill (CD-20, CD-22, and CD-23), the Basalt Aquifer encountered at Monitoring Well Location CD-20 was the only hydrogeologic unit with sufficient transmissive properties and available head for Phase I evaluation as a potential East System groundwater extraction source in this area.

The Basalt Aquifer appears to be of limited areal extent, as shown on Figure ER-4.11. The Basalt Aquifer is confined on top by (weathered or unweathered) Latah Aquitard and below by (unweathered) Latah Aquitard. The depth to the top of the Basalt Aquifer generally decreases from west to east, and the aquifer generally conforms to the surface of the Latah Aquitard. The Basalt Aquifer is present at about 180 ft BGS at the Landfill (Monitoring Well Location CD-4), and about 100 ft BGS at the (b) (6) private well located about 1,800 ft east of Monitoring Well Location CD-4.

As previously discussed in Section 4.1.2.3, the thickness of the Basalt Aquifer varies from about 10 ft (at Monitoring Well Location CD-7) to about 40 ft (at Monitoring Well Location CD-20). As discussed in Section 4.1.2.3, its erratic lateral distribution and general conformance to the surface topography of the Latah Aquitard suggests that the Basalt Aquifer may be a sequence of basalt blocks rather than a continuous unit.

Boundary conditions for the Basalt Aquifer are poorly defined. However, water level data (as shown on Figure ER-4.20) suggests that groundwater flow in the Basalt Aquifer is controlled by the contacts of the Basalt Unit with adjacent units. Groundwater elevations in Monitoring Well CD-4L reflect Lower Sand/Gravel Aquifer water levels, while water elevations at other well locations [CD-20, and (b) (6) [redacted] appear to reflect water levels of the adjacent Latah Aquitard.

Groundwater flow in the Basalt Aquifer is generally to the west, as shown on Figure ER-4.20. However, insufficient data are available to develop water elevation contours independently for the Basalt Aquifer.

4.2.5.2 Aquifer Parameters

Aquifer parameters for the Basalt Aquifer are based on the results from the CP-E2 APT and are summarized in Table ER-4.1. A description of the data analyses, along with data plots and calculations, are included in Appendix E. Data analyses indicate a T and K of about 25 ft^2/day and 0.7 ft/day, respectively. Because fracture flow predominates in the Basalt Aquifer, the estimated hydraulic conductivity represents a bulk value and the hydraulic conductivity of specific fractures zones may be significantly higher. Storativity for the Basalt Aquifer is estimated to be about 0.01. Based on the estimated hydraulic conductivity, a horizontal hydraulic gradient estimated from Figure ER-4.20, and an effective bulk porosity of 0.1 the average linear velocity is estimated to be about 0.4 ft/day for the Basalt Aquifer, as shown in Table ER-4.2.

Because Pilot Well CP-E2 was completed open-hole in the Basalt Aquifer, well efficiency was not estimated. Additionally, the analytical methods used for estimating aquifer transmissivity are based on fracture flow and cannot be used for estimating the radius of influence of the well. It should be noted that aquifer response to pumping at Pilot Well CP-E2 was observed in another well completed in the Basalt Aquifer (Monitoring Well CD-7L), but groundwater elevation data for a Basalt Aquifer well located about 1,200 ft east of Pilot Well CP-E2 (Goodwin) are ambiguous with respect to pumping response during the CP-E2 APT.

The T and K values estimated for the Basalt Aquifer are greater than an order of magnitude lower than those estimated in the RI (based on the (b) (6) [redacted] Well in the RI). Reevaluation of the data used for the RI analysis indicate a T and K of 26 ft^2/day and 1 ft/day, respectively, for the (b) (6) Well, which is similar to the aquifer parameters estimated from the CP-E2 APT. Re-evaluation of the RI data is presented in Appendix E.

The low transmissivity and slow recharge of the Basalt Aquifer is very significant to Phase II design, as it limits the practicability of groundwater extraction east of the Landfill. This is further discussed in Section 5.0.

4.2.6 <u>Conceptual Hydrogeologic Model</u>

A number of hydrogeologic boundary conditions converge in the immediate vicinity of the Landfill:

- The Lacustrine Aquitard pinches out, eliminating the hydraulic separation between the Upper and Lower Sand/Gravel Aquifers
- The Lower Sand/Gravel Unit transitions from unsaturated (to the east) to the primary regional aquifer (to the west)
- A lobe of the Latah Aquitard extends (westerly) into the Lower Sand/Gravel Aquifer, creating an east/west trending groundwater divide near the south edge of the Landfill.

These converging boundary conditions control migration of groundwater (and contaminants) from the Landfill into, and within, the Lower Aquifers.

Groundwater (from beneath the Landfill) enters the unsaturated Lower Sand/Gravel Unit either by direct infiltration through discontinuities in the Lacustrine Aquitard, or by lateral flow over the eastern edge of the Lacustrine Aquitard. Groundwater migrates vertically within the Lower Sand/Gravel Unit until contacting the upper surface of the Latah Aquitard. Groundwater then flows (as perched groundwater) along the Lower Sand/Gravel Unit and Latah Aquitard contact until it enters the Lower Sand/Gravel Aquifer (Northern or Southern) Flow Regime. A conceptual model of these groundwater flow characteristics is shown on Figure ER-4.22.

This conceptual model does not explain the migration of Constituents of Concern significant distances to the east (and northeast) of the Landfill observed during the RI and during Phase I (as described in Section 4.3). Two groundwater migration mechanisms appear to offer the greatest potential for explaining this apparent contradiction between groundwater flow and contaminant migration: 1) lateral flow to the east along a thin (and undetected) extension of the Lacustrine Aquitard; or 2) induced upgradient and cross gradient groundwater flow resulting from private well pumping of the Basalt Aquifer and (possibly) higher-permeability strata in the Latah Aquitard.

Although mechanism 1) is self explanatory, mechanism 2) warrants additional discussion. Groundwater flow within a basalt aquifer is often characterized by high T and low S within individual fractures or fracture zones. This allows groundwater to migrate large distances (hundreds or thousands of feet) when subjected to steep natural or induced hydraulic gradients. Such a mechanism may have resulted from prolonged private well pumping east of the Landfill causing a reversal in horizontal gradient, inducing groundwater flow from the Landfill vicinity to the east. Although the bulk transmissivity in the Basalt Aquifer is low and storativity relatively high (as described in Section 4.2.5.2), hydraulic properties for individual fractures or fracture zones may vary significantly from the bulk aquifer properties, allowing this migration mechanism to occur.

Figure ER-4.23 presents a conceptual model of these two potential groundwater infiltration and migration pathways. These potential pathways are further discussed in Section 4.3, within the context of the groundwater quality data.

4.3 GROUNDWATER QUALITY

The primary purpose of this section is to characterize VOC distribution and migration characteristics within the Site groundwater flow system. The VOC distribution and migration characteristics significantly impact Phase II design, effecting the location and screened interval of the interception/extraction systems and controlling the influent design concentrations for the treatment system. Typical values for physical and chemical properties of the Constituents of Concern that impact their migration and treatability characteristics are provided in Table ER-4.3.

A secondary purpose of this section is to characterize general water quality for constituents other than VOCs to assess their impact on treatment system design and to identify trends in general groundwater quality (if any) that may distinguish one aquifer (or aquifer zone) from another. In addition to VOC analyses, selected groundwater samples were analyzed for treatment system design parameters (iron, manganese, and hardness); landfill leachate parameters (cadmium, chloride, sulfate, nitrate/nitrite, TDS, TOC, total organic halides (TOX), and chemical oxygen demand (COD)]; and general water quality parameters (magnesium, calcium, potassium, and sodium). For all samples, pH, temperature, and conductivity were measured in the field. These Phase I groundwater quality data are tabulated in Appendix F.

Groundwater quality in the following subsections is discussed in terms of the Upper and Lower Aquifers. As previously described, the Lower Aquifers include the Lower Sand/Gravel Aquifer, the Latah Aquitard, the Basalt Aquifer, and Granite Aquitard. The Upper Aquifers include the Upper Sand/Gravel Aquifer, the shallow sand interbeds of the Lacustrine Aquitard, and the Fluvial Aquifer.

4.3.1 VOC Distribution

Two rounds of groundwater samples (winter to spring 1990 and winter to spring 1991) were collected and analyzed from all Phase I monitoring wells, and selected monitoring wells constructed prior to Phase I. One round of groundwater samples was collected from selected monitoring wells constructed prior to Phase I, and selected private wells. Analyses for the initial round of groundwater samples collected during Phase I (for all wells sampled) included the full suite of EPA Method 8010 VOCs. VOCs detected in the groundwater in both the Upper and Lower Aquifers include the Constituents of Concern (TCA, DCE, DCA, TCE, MC, and PCE); chloroform; 1,2-dichloroethane; 1,2-dichloropropane; and trichlorofluoromethane (Freon). Vinyl chloride and dichlorodifluoromethane were detected, but only in the Lower Aquifers. The compounds detected, other than the Constituents of Concern, were present in low concentrations in a limited number of wells, and are summarized in Table ER-4.4. Consequently, VOC analysis for the second round of groundwater samples was limited to the Constituents of Concern, as provided for in the Quality Assurance Project Plan (Landau Associates 1989c; page FS-3-2). The following discussions of VOC distribution are also limited to Constituents of Concern. A tabulation of analytical results for all volatile organic compounds detected during Phase I is provided in Table F-1 of Appendix F.

The distribution of the Constituents of Concern discussed in the following subsections is based on both rounds of groundwater quality data collected during Phase I and data from the Landfill Domestic Well Sampling Program (Domestic Well Program). Data from the Domestic Well Program were collected independent of Phase I activities by other parties, and QA/QC for these data were not evaluated and are not discussed in this report. However, the Domestic Well Program data appear to be consistent with Phase I data and are considered adequate for the purposes of their application herein. Domestic Well Program data used in characterizing constituent distributions are provided in Appendix F.

4.3.1.1 Upper Aquifers

The areal distribution of the Constituents of Concern for the Upper Aquifers is shown on Figures ER-4.24 through ER-4.29. In general, the areal extent of the Constituents of Concern (above detection) is similar to that shown in the RI, except the distribution extends farther to the southeast and extends west of the Upper Sand/Gravel Aquifer into the Fluvial Aquifer.

The areal extent over which Constituents of Concern exceed the Performance Standards is much smaller than was characterized during the RI. Significant exceedances of the Performance Standards identified during Phase I are limited to DCE near the leading edge of the plume (see Figure ER-4.25), although historic water quality data for upgradient domestic wells suggest concentrations of TCA significantly in exceedance of the Performance Standards are also present in this area (see TCA data for Irgens private well on Figure ER-4.38). Conversely, only minor exceedances of the TCE Performance Standard were observed near the leading edge of the plume (see Figure ER-4.27), and minor exceedances of PCE and MC were observed near the Landfill (see Figures ER-4.29 and ER-4.28, respectively). No exceedance of the DCA Performance Standard was detected (see Figure ER-4.26).

Minor exceedances of TCA and DCE Performance Standards were also observed southeast of the Landfill in shallow sand interbeds of the Lacustrine Aquitard at Monitoring Well CD-23B1) (see Figures ER-4.24 and ER-4.25, respectively). As discussed in Sections 4.1 and 4.2, the Lacustrine Aquitard contains a number of fine sand interbeds that appear to be hydraulically connected to the Upper Sand/Gravel Aquifer.

4.3.1.2 Lower Aquifers

The areal distribution of the Constituents of Concern for the Lower Aquifers is shown on Figures ER-4.30 though ER-4.35. The areal extent of the Constituents of Concern (above detection) has expanded slightly to the south, west, and northeast from that shown in the RI. In general, the maximum concentrations observed during Phase I are similar to those detected during the RI. However, maximum detected concentrations for TCA decreased somewhat, from about 4,800 ppb in the RI to about 2,800 ppb during Phase I. TCA is the most widely distributed Constituent of Concern, and was detected throughout the zones within the Lower Aquifers where the Constituents of Concern are present (see Figure ER-4.30). DCE and DCA exhibit a similar, although more limited, areal distribution (see Figure ER-4.31 and ER-4.32, respectively). TCE is limited to a smaller area in the Landfill vicinity and exhibits its highest concentrations south of the center of the Landfill (see Figure ER-4.33). MC is also limited to the Landfill vicinity, but is only present from about the center of the Landfill north (see Figure ER-4.34). PCE is limited to the immediate Landfill vicinity (see Figure 4-35).

The distribution of TCE and MC appear to support the hydrogeologic conceptual model (presented in Section 4.2.6) of an east-west groundwater divide in the Lower Aquifers, and a Southern and Northern Flow Regime for the Lower Sand/Gravel Aquifer. MC is only present in the Northern Flow Regime, and TCE is primarily distributed in the Southern Flow Regime. Additionally, the highest concentrations of TCA are also in the Northern Flow Regime. This distribution also suggests a different disposal history for MC and TCE. MC appears to have been disposed of in such a manner that it entered the Lower Aquifers in the central or northern portions of the Landfill. TCE was disposed of such that it entered the Lower Aquifers in the southern portion of the Landfill or at another location (or locations) south of the Landfill.

The vertical distribution of the Constituents of Concern was evaluated for well clusters constructed during Phase I that have more than one well screened in the Lower Sand/Gravel Aquifer. The distribution of TCA for these wells is shown on Figure ER-4.36 (other Constituents of Concern, when present, show a similar distribution). As the figure indicates, concentrations are higher in the upper zones of the Lower Sand/Gravel Aquifer and correlate well with the vertical distribution of groundwater flow suggested by the vertical gradients on Figure ER-4.18. It should be noted that Monitoring Well CD-40C1 is designated a Lower Sand/Gravel Aquifer well but, for constituent distribution and mass estimate purposes, it is considered a Fluvial Aquifer well.

4.3.2 Constituent Migration

Migration trends for the Constituents of Concern were evaluated based on data collected during Phase I, data from the RI, and data from the Domestic Well Program. Constituent migration characteristics evaluated include rate and direction, and retardation factors.

4.3.2.1 Upper Aquifers

Constituent migration in the Upper Aquifers appears to largely conform to groundwater flow, and is generally to the south, deflecting to the southeast near the leading edge of the plume. This migration path appears to conform to the apparent north-south trending trough at the top of the Lacustrine Aquitard identified in Sections 4.1 and 4.2. There is a secondary component of constituent migration to the west (into the Fluvial Aquifer), also apparently controlled by groundwater migration. Although not in direct hydraulic connection, the Upper Sand/Gravel Aquifer, and possibly the shallow sand interbeds of the Lacustrine Aquitard, appear to recharge the Fluvial Aquifer through springs and possibly subsurface interflow.

Because the Constituents of Concern have not migrated further west than Highway 2 in the Lower Sand/Gravel Aquifer (see Figures ER-4.30 through ER-4.35), the Upper Sand/Gravel Aquifer appears to be the source of the Constituents of Concern detected in the Fluvial Aquifer. Constituent migration in the Fluvial Aquifer appears to be toward the west, to the Little Spokane River, although only limited hydrogeologic and chemical data are available for this unit.

Changes in TCA concentration over time were evaluated for a number of Upper Sand/Gravel Aquifer wells and springs. As shown on Figures ER-4.37 and ER-4.38, TCA concentrations have decreased significantly with time in the Upper Sand/Gravel Aquifer, both near the Landfill and a short distance behind the leading edge of the plume. This migration pattern suggests that the constituent source for the Upper Sand/Gravel Aquifer is reduced and concentrations should continue to decline. It should be noted that concentrations for the Friedrichsen Well (Figure ER-4.38) indicate a peak and subsequent decrease in concentration. Upgradient water quality data (Wells CD-30A, CD-33A, and CP-S1) indicate this migration pattern probably represents a leading edge "stringer" in advance of the main body of the plume, and concentrations are expected to increase at this location in the future.

Figures ER-47 and ER-4.39 show TCA concentrations versus time for Upper Sand/Gravel Aquifer in the Landfill vicinity and for springs that discharge to the Fluvial Aquifer, respectively. TCA concentrations in these areas have significantly decreased with time, which suggests that the source of the Constituents of Concern for the Fluvial Aquifer has decreased. Ultimately, this decrease in concentration should be reflected in the Fluvial Aquifer, although it may be a significant period of time (years) before a clear trend in constituent concentrations is discernible for the Fluvial Aquifer.

The migration rate for TCA was evaluated by examining the time between concentration peaks at different wells (migration rates for other constituents were not evaluated due to data limitations). Based on this evaluation, the TCA migration rate ranges from about 2.5 to 4.3 ft/day for the Upper Sand/Gravel Aquifer, as shown in Table ER-4.4. This constituent velocity translates to a retardation factor of 1.4 to 2.4 for TCA if a groundwater flow velocity of 6 ft/day is assumed, and compares well with the typical retardation factor for TCA of 1.8 provided in Table ER-4.3. This method of estimating constituent retardation is only approximate, but it is useful in evaluating the consistency of constituent migration rates and estimated groundwater flow velocities. Insufficient data are available to evaluate contaminant migration rates for the Fluvial Aquifer.

4.3.2.2 Lower Aquifers

The Constituents of Concern have migrated in the direction of groundwater flow in the Lower Aquifers. However, they have also migrated significant distances upgradient and cross gradient to the direction of groundwater flow. These anomalous migration characteristics were explained in the RI as being the result of the Constituents of Concern migrating in the form of dense nonaqueous phase liquids (DNAPLs). Site history and data collected during Phase I do not support anomalous constituent migration by this DNAPL theory for the following reasons:

- Section 5.4.1 of the RI (Golder Associates 1987a) indicates that the solvents were disposed of by pouring the solvent mixtures in trenches that were covered soon after disposal. This method of disposal results in a disperse source, which does not provide the source mechanism typically required for accumulation of large masses of DNAPL at a single location (in pools) or for DNAPL migration a significant distance from the Landfill, given a limited source and thick vadose zone (conditions identified for the Landfill).

- Dissolved constituent concentrations in the vicinity of a large DNAPL source within a groundwater flow system would be expected to remain at levels of about 1 percent of the solubility of the constituent in water for tens of years. This would be about 9,000 ppb for TCA, based on the solubility for TCA provided in Table ER-4.3. Constituent concentrations in the Upper Sand/Gravel Aquifer near the Landfill have decreased to 39 ppb or less (see Figure ER-4.24), which is significantly below this anticipated TCA concentration in the vicinity of a DNAPL pool. This discrepancy between measured concentration and anticipated concentration in the vicinity of a DNAPL pool suggests that a continuing source of contamination (DNAPL pool) in the Upper Sand/Gravel Aquifer is not present.

- Dissolved constituent concentrations in the vicinity, and downgradient, of a DNAPL source would be expected to remain elevated for tens of years because of the relatively low water solubility of most chlorinated solvents in water. However, constituent concentrations to the east, north, and northeast of the Landfill in the Lower Aquifers (upgradient and cross gradient to groundwater flow) have decreased significantly with time to levels that are well below 1 percent of the solubility for a given constituent, as shown on Figures ER-4.40 through ER-4.42 (see Table ER-4.3 for Constituents of Concern solubilities). This decrease in concentration suggests that DNAPLs are not present within these areas of the Lower Aquifers.

- The vertical distribution of contaminants in the Lower Sand/Gravel Aquifer indicate higher concentrations near the top of the aquifer and conform to groundwater flow patterns, contrary to what would be expected if DNAPLs were present in the Lower Sand/Gravel Aquifer.

- As shown on Figure ER-4.5, the low-permeability contact along which DNAPL migration would occur east of the Landfill (the Upper or Lower Sand/Gravel-Latah Formation interface) slopes downward to the west, opposite from the direction that would be expected to cause DNAPL migration to the areas exhibiting anomalous constituent migration trends.

This is not to suggest that DNAPLs are not present at the Landfill, only that there is little evidence that DNAPLs are present in the groundwater flow system and are causing the observed constituent migration patterns. As discussed in Section 4.3.3, there is a significant probability that DNAPLs are present in the refuse and in the vadose zone underlying the refuse at the Landfill.

As described in Section 4.2.6, the most probable mechanisms for causing the anomalous constituent migration observed in the Lower Aquifers are influences from private well pumping and migration of contaminated groundwater along an extension of the Lacustrine Aquitard to the east, or a combination of the two. The upper surface of the Latah Aquitard is at a higher elevation at some locations east of the Landfill (where groundwater contamination is present) than the elevation of the Lacustrine Aquitard at the Landfill. Consequently, migration to the east along an extension of the Lacustrine Aquitard cannot account for the full extent of contaminant distribution in the Lower Aquifers. The significant decrease in constituent concentrations east of the Landfill that occurred subsequent to many of the residents' connection to alternative water (causing a cessation or reduction in private well pumping) support pumping well influences as the primary cause of anomalous constituent migration to the east of the Landfill. The north-south cross gradient spread of contamination in the Lower Aquifers east of the Landfill may also result from pumping well influences. Also, it is possible that preferred migration pathways exist in the Lacustrine Aquitard upper erosional surface, and that discharge to the Lower Aquifers from these pathways (if they exist) may contribute to this north-south contaminant spread.

It is anticipated that contaminant migration in the Lower Aquifers east, northeast, and southeast of the Landfill will revert to directions consistent with groundwater flow when no longer influenced by private well pumping. Although many private wells east of the Landfill are no longer used, some wells are used and may continue to impact contaminant migration in this area.

As shown in Table ER-4.4, the migration rate for TCA in the Lower Sand/Gravel Aquifer southwest of the Landfill is estimated to be 0.5 ft/day or less. Due to data limitations, this migration rate was estimated based on initial rather than peak concentrations. Further, TCA was detected in the upgradient well [CD-2(L)] at the time of construction, so the migration rate represents a maximum value. This migration rate translates to a retardation factor of at least 1.2, based on an average linear velocity for groundwater in the Lower Sand/Gravel Aquifer of 0.6

ft/day. This retardation factor is in reasonable agreement with the typical retardation value of 1.8 provided for TCA in Table ER-4.3, and indicates that estimated groundwater flow velocities for the Lower Aquifer are consistent with constituent migration rates.

4.3.3 Constituent Mass

The total mass of each Constituent of Concern present in the Upper and Lower Aquifers was estimated based on Phase I data and data from the Domestic Well Program. The primary purpose of this estimate is to provide the data needed to estimate the average mass flux to the Phase II treatment system for evaluation of the need for air emissions abatement which is addressed in Section 4.5. These estimates are presented in Table ER-4.5, along with estimates from the RI of the total mass of TCA and MC disposed of at the Landfill. The procedures used to estimate constituent mass are provided in Appendix G. The estimated mass of TCA in the groundwater system represents a significant percentage (27 percent) of the estimated disposed mass. However, the estimated mass for MC represents only about 3 percent of the estimated disposal mass.

Section 1.3 of the RI (Golder Associates 1987a) indicates MC was disposed of concurrently with TCA by the Key Tronic Corporation. Because MC is much more soluble in water than TCA and does not partition onto soil to as great a degree as does TCA (see Table ER-4.2 for constituent properties), MC should migrate in the subsurface at a more rapid rate than TCA. Thus, the total mass of MC measured in the groundwater flow system should be greater than the total mass of TCA, based on the estimated disposed masses provided in the RI (and shown in Table ER-4.5). This may indicate that the estimated disposal mass for MC is significantly overestimated, or the disposal mass for TCA is significantly underestimated. Also, significant masses of TCA and MC may remain in DNAPL form in the Landfill refuse and the vadose zone underlying the Landfill. As discussed in Section 4.3.2.2, available data do not support the presence of DNAPLs in the groundwater flow system.

4.3.4 General Groundwater Quality

General groundwater quality in the Landfill vicinity was evaluated so its impact on Phase II treatment system design could be assessed. Additionally, variations in groundwater quality between the Upper and Lower Sand/Gravel Aquifers, and depth zones within the Lower Sand/Gravel Aquifer, were evaluated to determine if any trends in general groundwater quality were evident. General water quality analysis results are summarized in Table ER-4.5. Complete results are provided in Appendix F.

Monitoring Wells CD-30A (Upper Sand/Gravel Aquifer) and CD-21C1 and CD-21C3 (Lower Sand/Gravel Aquifer) are located in areas where the presence of Landfill leachate would be anticipated. As shown in Table ER-4.5, chloride, hardness, TDS, TOX, calcium, and conductivity are slightly to moderately elevated for Monitoring Wells CD-20C1 and CD-30A, and pH is somewhat lower at these two locations. Other general groundwater quality parameters do not exhibit significant variation between wells.

The most significant impact of general groundwater quality on Phase II treatment system design is elevated hardness (also evident in terms of elevated calcium for Monitoring Well CD-20C1). The impact of hardness on Phase II design (in terms of chemical scale formation) is discussed in Section 4.4.

Two easily measured field parameters, pH and conductivity, were compared to TCA concentration to determine if a correlation exists. As shown on Figure ER-4.43, a general (although inconsistent) trend of decreasing pH with increasing TCA concentration exists. Figure ER-4.44 shows a somewhat clearer trend of elevated conductivity co-occurring with elevated TCA concentration. In most instances, TCA concentration exceeds 200 ppb when conductivity exceeds 500 μmhos/cm. TCA (and the other Constituents of Concern) in dissolved form should not significantly impact groundwater pH or conductivity. Therefore, the observed trends are probably the result of the Constituents of Concern migrating with general Landfill leachate (which can impact pH and conductivity). Because pH and conductivity are easily monitored in the field, it is anticipated that these trends can be utilized during Phase II well construction to assist in the selection of monitoring and extraction well screened intervals. However, the scatter in data indicate these parameters can be used only as general guidance during drilling, not as specific criteria.

The relative percentage of common anions and cations present in ground are often compared to evaluate trends in general groundwater quality; these data are typically evaluated in a "Piper Diagram", as shown on Figure ER-4.45. A Piper Diagram is created as follows: 1) the relative percentage of cations (Mg, Ca, Na+K) are plotted on the triangle in the lower left corner of the figure; 2) anions (SO_4, $Cl+NO_3$, CO_3+HCO) are plotted on the triangle in the lower right corner of the figure; and 3) the anion and cation plots for a given well are projected to their intersection on the combined diamond plot in the center of the figure, resulting in groupings of wells with similar water quality characteristics.

As shown on Figure ER-4.45, all wells plot approximately the same (as calcium carbonate-rich waters). No strong trends or distinctions in general groundwater quality are apparent between the Upper and Lower Sand/Gravel Aquifers or between zones in the Lower Sand/Gravel Aquifer. These data suggest that groundwater in the Upper Sand/Gravel Aquifer and the Lower Sand/Gravel Aquifer originates from the same source or migrate through similar geologic media.

4.4 TREATABILITY STUDY DATA EVALUATION

The Phase I Treatability Study was conducted to obtain air stripping performance data under actual field conditions to optimize the design of the Phase II treatment system. Specific Treatability Study goals included:

- Evaluation of the capability of air stripping to achieve Project effluent Performance Standards
- Determination of the effect of design variables (hydraulic loading, air-to-water ratio, packing size, packing height) on the performance of the air stripping system
- Assessment of the need for mineral and biological scale control
- Collection of field data to calibrate the model to be used for design of the Phase II air stripping system.

Four variables are available in designing a packed tower air stripping system: 1) hydraulic loading (and, thus, the tower diameter); 2) air-to-water ratio; 3) packing shape and size (and, thus, packing surface area); and 4) packing height. Each of these variables were evaluated as part of the Treatability Study testing program. The following subsections discuss Treatability Study results, design variable analysis, and the methodology for Phase II design.

4.4.1 Data Collected From Treatability Study

Seventeen trial runs were conducted during the Treatability Study. Sixteen of the runs were conducted to evaluate design parameters. The seventeenth run was conducted to evaluate the consistency of influent and effluent concentrations at constant operating conditions. The parameters that were varied during the Treatability Study were packing size (2- and 3.5-inch diameter), hydraulic load (150 and 200 gpm nominal flow rates), and air flow rate (variety of flow rates between 1,330 and 2,170 cfm). Evaluation of constituent removal versus packing height was also conducted for each run. Each trial run lasted approximately two to three hours. Three influent and three effluent samples were collected periodically for each trial run, during collection of water samples from 10, 20, 30, and 40 feet above the base of the tower. Quality assurance (duplicate and blank) samples were also collected and are discussed in Section 3.8. Table ER-4.8 presents the results of chemical analyses of samples collected during the Treatability Study.

<u>Methylene chloride concentrations provided in Table ER-4.6 were adjusted for blank contamination for use in Treatability Study analyses. Blank correction is not an EPA-accepted practice for many data uses because errors associated with low level concentrations may be compounded. However, methylene chloride laboratory contamination is ubiquitous, and blank correction is a reasonable and practical method of addressing this problem for the purposes of the Treatability Study. It should be recognized that use of these data for other purposes may not be appropriate. As a result, methylene chloride data were also validated using EPA functional guidelines, and the data validated in this manner are provided in Table F-4 of Appendix F.</u>

Review of the data show that TCA, DCE, DCA, TCE, and PCE were all removed to well below effluent Performance Standards in all trial runs. Methylene chloride, which is the most difficult to remove of the Constituents of Concern, approached effluent Performance Standards in several of the trial runs.

Analyses were also performed for hardness, alkalinity, TDS, pH, and conductivity to evaluate the scaling potential of the water. A trend of reduction in hardness, alkalinity, and TDS from the influent to the effluent of the air stripping tower indicates that scale formation is occurring. A trend of increase in pH from influent to effluent is representative of transfer of carbon dioxide from the water to the air.

Both air and water temperatures were recorded during the trial runs. Groundwater temperatures varied slightly, ranging from approximately 10-12 degrees centigrade (C), and little change in water temperature was observed between stripping tower influent and effluent. These groundwater temperatures represent the anticipated range of operating conditions for the Phase II system. Influent air temperatures ranged from -2 to 17 degrees C, and effluent air temperatures approached the groundwater temperature. These air temperatures represent the lower range of operating conditions for the full scale system; however, these are not the lowest temperatures anticipated. Even at subzero air temperatures, the water temperature remains relatively constant throughout the tower. This is important because lower water temperatures would cause reduced removal efficiency due to lowering of the vapor pressure of the constituents.

Two groundwater sources were used for the test: 1) Pilot Well CP-E1, and 2) combined flow from Pilot Wells CP-E1 and CP-E2. CP-E2 contributed a maximum of about 20 gpm to the influent water, with CP-E1 contributing about 130-180 gpm, depending on the hydraulic loading rate tested. Due to the low percentage of flow contributed by CP-E2, the only significant change between the two sources was higher TCE concentrations. Groundwater concentrations from CP-E1 decreased between the initial trial runs to later trial runs. This decreasing trend could be due to the movement of lower concentration groundwater toward the well as extraction progressed. Trial No. 17 showed influent and effluent concentrations for each constituent to be relatively constant within this trial. This short-term consistency of influent and effluent concentrations was noted in most, but not all, trials.

4.4.2 Mass Transfer Design Variables

The steady-state rate of mass transfer of a solute (TCE, and other Constituents of Concern) across an air/water interface within a unit volume of packing can be expressed as

$$J = K_l \underline{a} (C^e_l - C_l)$$

in which J = rate of mass transfer of solute per unit volume of packing, K_l = the overall mass transfer coefficient, $\underline{a}$ = the interfacial area per unit volume of tower, C^e_l = concentration of solute in the water phase which would be in equilibrium with the existing air phase concentration, and C_l = average concentration in the water phase. The rate of mass transfer, J, integrated over the total height of a packed column will define the overall mass transfer capacity of the tower.

The mass transfer coefficient (K_l) is dependent on the water and air flow rates, the physical properties (i.e., viscosity, density, diffusivity) of the water, solute and air, and the size and shape of the packing used. The interfacial area ($\underline{a}$) is dependent on the size and shape of the packing used, the water flow rate, and physical properties of the water and packing. Because both K_l and $\underline{a}$ are dependent on the water and air loading rate and the size and shape of the packing, their product, $K_l\underline{a}$ (called the overall mass transfer rate constant), is used to evaluate the rate of transfer (from water to air phase) at a given set of operating conditions.

The equilibrium water phase concentration (C^e_l) for dilute solutions is described by Henry's Law as

$$C^e_g = H\, C^e_l$$

where C^e_g is the air phase concentration in equilibrium with a given water phase concentration C^e_l, and H is the Henry's Law Constant. The Henry's Law Constant is a unique physical property of a particular solute and is strongly dependant on the temperature of the water containing the solute. The Henry's Law Constant increases with increasing temperature. As shown in Table ER-3, methylene chloride has a significantly lower Henry's Law Constant than the other Constituents of Concern. Consequently, methylene chloride is the most difficult compound of the Constituents of Concern to remove by air stripping. Providing fresh air to the interfacial area keeps the air phase concentration low, driving the equilibrium water phase concentration down and, thus, creating a driving force for mass transfer.

As the physical properties of the water, solute, and air are constant at a given temperature and pressure, the variable parameters which can be adjusted to increase mass transfer of a solute from the water to the air phase are the water and air flow rates, the size and shape of the packing, and the height of the packing available for mass transfer. The air flow rate is evaluated as the ratio of the air-to-water flow rates, to correlate the air flow rate available for mass transfer per unit water flow rate. Each of these variables were evaluated in the Treatability Study.

4.4.3 Air Stripping Performance Evaluation

The results of the Treatability Study show that air stripping can be successful in achieving the effluent Performance Standards. As will be shown in the subsequent subsections, selection of appropriate design parameters for the Phase II system is predicted to achieve the effluent goals.

The performance of the pilot air stripping system was within reasonable expectations based on documented accuracy of the design equations used in predicting air stripping system performance. A calculation error in the prediction of performance for the 3.5-inch packing occurred during Treatability Study design and resulted in higher than expected effluent concentrations for those trial runs; this calculation error was not made in the predictions of the 2-inch packing performance. The error consisted of using incorrect packing material properties, was detected during Treatability Study analysis, and was corrected for the data analysis presented in this report. Because the Treatment System had sufficient air flow capacity, air flow rates were increased to meet Phase I effluent discharge criteria (Evaluation Criteria), and the initial design error did not prevent successful completion of the Treatability Study.

4.4.4 Design Variable Analysis

As shown in Table ER-4.8, the effluent Performance Standards were readily achieved at all Treatability Study operational settings for all Constituents of Concern, except methylene chloride. Consequently, the tower design parameters will be controlled by the requirements for methylene chloride removal. The evaluation of design parameters presented below is discussed in terms of the removal efficiency achieved for methylene chloride.

Comparison of the data in Table ER-4.8 reveal data points which were not consistent with the rest of the data set, possibly due to short-term fluctuations in constituent concentration. These data are included in Table ER-4.8, but were not used for data analysis.

Table ER-4.9 presents a tabulation of the design variables and test results evaluated in the Treatability Study, <u>and</u> average values of the influent and effluent concentrations of methylene chloride which are used during the analysis of the design parameters presented below. The air-to-water ratios presented in Table ER-4.9 were calculated on a volume-to-volume basis. The calculated mass transfer constants presented in Table ER-4.7 were calculated using the VOLSTRIP model, which is based on the equations presented in Appendix H. The measured mass transfer coefficients were calculated from the Treatability Study data using the equations presented in Appendix H. Removal efficiencies were calculated based on average influent and effluent concentrations of methylene chloride.

Comparison of the methylene chloride removal efficiency between trials displays the effects of the design variables on system performance. For example, the removal efficiency achieved by the 2-inch diameter packing exceeded that of the 3.5-inch diameter packing at all operational settings. Calibration of the model to be used in design of the Phase II system involves comparing the theoretically calculated mass transfer rate constants versus those measured in the Treatability Study. The measured mass transfer rate constant will be used for modeling and scale up of the Phase II system.

In reviewing the mass transfer rate constants presented in Table ER-4.9, the calculated values for the 3.5-inch packing were somewhat lower than the measured values, indicating that the 3.5-inch packing performed slightly better during the Treatability Study than predicted by the model. The calculated mass transfer rate constants for the 2-inch packing are higher than measured values, indicating somewhat poorer performance than predicted.

The required removal efficiency for the Phase II system is anticipated to be about 99.6 percent (based on the FS methylene chloride influent concentration estimate of 560 µg/l, and the effluent Performance Standard of 2.5 µg/l). Treatability Study removal efficiencies for methylene chloride ranged from 98.96 to 99.85 percent, and 8 of the 16 Treatability Study trials achieved this target removal efficiency. The following subsections discuss Treatment System response to variation of the design parameters (hydraulic loading, air-to-water ratio, packing diameter, and packing height).

4.4.4.1 Hydraulic Loading

The hydraulic loading has two conflicting effects on removal efficiency: 1) at a higher hydraulic loading rate, a greater interfacial surface area exists per unit volume of packing for mass transfer to occur [consequently increasing the mass transfer rate constant ($K_l\underline{a}$)], and 2) at a higher hydraulic loading rate, there is a greater mass of solute per unit time which requires removal, and the liquid film thickness may increase, both potentially reducing removal efficiency for a given air flow rate and packing volume. Also, an excessively high hydraulic loading can lead to flooding of the packing, causing channeling and/or hold up of the liquid in the tower, which results in lower removal efficiency.

Nominal hydraulic loadings of 150 and 200 gpm were evaluated in the Treatability Study. The design variable analysis data for each group is presented in Table ER-4.10. Figures ER-4.46, ER-4.47, and ER-4.48 present graphs of hydraulic loading versus methylene chloride removal efficiency for the 2- and 3.5-inch packings at different air-to-water ratios. Neither of the packing sizes displayed significant change in removal efficiency with hydraulic loading, although a slight increase in removal efficiency with increasing hydraulic loading is apparent for 3.5-inch packing, and a slight decrease in removal efficiency may occur for the 2-inch packing. This indicates that both packings were operating in a range of acceptable hydraulic loading. The data also show the 2-inch packing consistently achieves higher removal efficiencies than the 3.5-inch packing over the range of hydraulic loadings tested, although removal efficiency of 3.5-inch packing approaches that of 2-inch packing at high hydraulic loadings and high air-to-water ratios.

4.4.4.2 Air-to-Water Ratio

Increasing the operating air-to-water ratio increases removal efficiency, until a point is reached where removal rates remain relatively constant. This point of diminishing return in removal efficiency with increased air to water ratio varies with packing type and size, and with the constituent to be removed.

Air-to-water ratios were varied from about 66 to 81 ($volume_{air}/volume_{water}$) for the Treatability Study, and were segregated for analysis into ratios of 60-70, 70-80, and over 80. The design variable analysis data for each group is presented in Table ER-4.11. Figure ER-4.49 presents a graph of air-to-water ratios versus methylene chloride removal efficiency for the 2- and 3.5-inch packings at a hydraulic loading of approximately 150 gpm, and Figure ER-4.50 presents the same comparison for a hydraulic loading of approximately 200 gpm.

The 2-inch packing displays no significant change in removal efficiency with variations in air-to-water ratio, whereas the 3.5-inch packing displays a general increase in removal efficiency with increasing air-to-water ratio. This is likely the result of the 3.5-inch packing not having achieved available mass transfer capacity; whereas, the 2-inch packing has approached its maximum removal efficiency (point of diminishing return) for the applied hydraulic loadings.

The data also show the 2-inch packing achieved higher removal efficiencies than the 3.5-inch packing over the range of air-to-water ratios tested. However, the efficiency of the 3.5-inch packing begins to approach that of the 2-inch packing at the upper end of the air-to-water ratios tested, indicating required (Phase II) removal efficiencies may possibly be achieved using 3.5-inch packing if higher air-to-water ratios (than used in the Treatability Study) are used.

4.4.4.3 Packing Diameter

The 2- and 3.5-inch diameter packings used in the Treatability Study were both manufactured by Jaeger Products, Inc. (Jaeger Tri-Packs), and have the configuration of a hollow, spherical shape with a network of ribs, struts, and drip rods. The 2-inch diameter packing provides 48-ft^2 surface area per cubic foot of packing, and the 3.5-inch diameter packing provides 38-ft^2 surface area per cubic foot of packing. Consequently, the 2-inch packing provides over 20 percent greater available interfacial surface area for mass transfer than the 3.5-inch packing for a given packed volume of tower, with an associated increase in removal efficiency expected (for the 2-inch packing) at a given air and hydraulic loading rate.

Table ER-4.12 presents a comparison of the packing performance over the range of design variables evaluated. This table shows that the 2-inch diameter packing achieved higher removal efficiencies than the 3.5-inch diameter packing at all operational settings. The average removal efficiency for all trial runs with the 2-inch packing was 99.76 percent, compared to 99.34 percent for the 3.5-inch packing.

This evaluation indicates that the interfacial surface area for mass transfer is a key design parameter for the Phase II system. Adequate surface area can be provided through selection of packing diameter, shape of packing, or the volume of packing in the tower. Many shapes and sizes of packing are available, each with different surface areas and mass transfer characteristics.

Consideration of plugging by chemical scale should also be factored in the packing selection, as discussed in Section 4.4.5 (i.e. the larger the packing, the less susceptible to plugging by scale).

4.4.4.4 Packing Height

Packing height is typically selected to achieve a target removal efficiency in concert with selection of other design variables. The limitation on tower height comes from the operating cost of pumping liquid to the top of the tower, the pressure drop of the air flow through the tower, as well as structural considerations.

Table ER-4.8 shows the concentrations of constituents at 10-foot vertical intervals in the tower. Target removals for all Constituents of Concern except methylene chloride were achieved within the 41.6-ft packed height of the Phase I air stripping tower. Review of the data for methylene chloride in Table ER-4.8 shows decreasing methylene chloride concentrations between the sample taken 10 ft above the bottom of the tower and the effluent sample. The average removal efficiency in the bottom 10 ft of packing for all runs was nearly 60 percent, indicating significant stripping of methylene chloride was occurring in the bottom section of the tower.

Figures ER-4.51 and ER-4.52 present methylene chloride concentration versus depth of packing traversed by the water for Trial Nos. 2 and 13, respectively. Similar methylene chloride concentration profiles for all trails are presented in Appendix H. Extrapolation of the methylene chloride concentration versus height data shown on these figures indicate that the effluent Performance Standards could be achieved by increasing the tower height. This suggests that methylene chloride will likely determine tower height for Phase II design if the influent design concentrations of methylene chloride for Phase II system are at the levels presently anticipated.

Evaluation of the figures in Appendix H indicate a trend of linearity in concentration with depth of packing (as plotted on semi-log scale) for trials with 150 gpm hydraulic loading. For trials with 200 gpm hydraulic loading, a divergence from linearity occurs in the lower 10 ft of packing, particularly in trials where the 3.5-inch packing was used. This indicates that the removal efficiency (concentration into section versus concentration out of section) remained relatively constant over the lower tower section at the 150 gpm loading. However, a drop in removal efficiency in the lower tower section was seen at the 200 gpm hydraulic loading. This analysis shows that a straight line correlation is inappropriate for representing the contaminant profile at the higher hydraulic loading rate, whereas a linear correlation appears to be valid at 150 gpm hydraulic loading. This relationship will be considered during design of the Phase II treatment system.

4.4.5 Chemical and Biological Scale Assessment

Observations of the Treatment System during the Treatability Study indicate significant chemical scale formation occurred on the packing and effluent piping. No biological scale formation was observed during the tests. Both of these findings were anticipated, based on the relatively high alkalinity and hardness (indicating high chemical scale formation potential) and relatively low total organic carbon content of the water (indicating low biological scale formation potential). Scale formation occurred on both the 3.5-inch and 2-inch packings.

During disassembly of the air stripping tower, chemical scale formation was found to be stratified within the tower packing, with increasing scale formation from the top to the bottom of the packing. The top 20 ft of packing showed only slight to moderate chemical scale formation, whereas the lower 20 ft of packing showed significant coverage of the packing with scale. In addition, settleable solids were observed in the effluent clearwell, with no solids detected in the influent groundwater.

Table ER-4.8 presents analytical data for the hardness, alkalinity, TDS, and pH at 10-ft vertical intervals within the air stripping tower. These data show a trend of decreasing hardness, alkalinity, and TDS, and increasing pH, from the top to the bottom of the tower.

The trend of increase in pH from influent to effluent is representative of transfer of carbon dioxide from the water to the air, which raises the pH. Alkalinity is primarily a measure of carbonate and bicarbonate levels of the water. Hardness is primarily a measure of the calcium, magnesium, and iron levels of the water. As the pH of the water rises, formation of calcium carbonate occurs causing scale formation. The decrease in hardness is caused by the precipitation of the calcium ions. The decrease in alkalinity is due to the lower solubility of carbonate at pH 8 to 9 compared to pH 6 to 7. This phenomena is borne out by the highest scale formation occurring in the lower 20 ft of packing where the pH has risen above 8. A decrease in TDS is indicative of removal of alkalinity and hardness through the air stripping tower.

4.4.6 Methodology For Phase II Design

Design of the Phase II treatment system is highly dependent on Phase II hydraulic loading and constituent influent concentrations (primarily methylene chloride). Phase II hydraulic loading and constituent concentrations are dependent on design of the South and West Phase II Interception Systems and the East Phase II Extraction System. Design of these interception/extraction systems will occur following review of this report by EPA and Ecology and approval to proceed with Phase II design. Thus, specific Phase II treatment system design parameters cannot be presented at this time.

The following subsections present the methodology for Phase II design. Treatment system design parameters will be provided in the Phase II Treatment and Discharge Plan.

4.4.6.1 Model Calibration

The results of the Treatability Study will be used as the design bases for the Phase II treatment system. Treatability Study trials performed using the operating conditions that most closely represent the operating conditions for the Phase II treatment system (when selected) will be used as the bases for design of the final system. The consistency and statistical accuracy of the data will also be considered and factored into selection of Phase II design parameters to provide a reasonable factor of safety. The Treatability Study results will be used to calibrate the design model to the mass transfer coefficients (described in Section 4.4.2) measured during the Treatability Study.

The design model is calibrated by modifying the mass transfer coefficient through the use of an accuracy factor, which converts the value of the predicted mass transfer coefficient to the measured value. Accuracy factors vary for different air stripping tower operating parameters, and are unique for each constituent evaluated. The model used for prediction of the performance of the pilot air stripping system (VOLSTRIP) is described in detail in Appendix H. Calculation of the measured mass transfer coefficient and the development of accuracy factors are also presented in Appendix H.

Appendix H presents a comparison of the spreadsheet printouts of model input and output for both the predicted tower performance and the calibrated performance using the accuracy factors generated from the pilot test for Trial No. 13. As these printouts indicate, the calibrated model accurately predicts the effluent concentrations obtained during the Treatability Study.

4.4.6.2 Design Parameter Selection

Design parameters for packed tower air stripping include: 1) hydraulic loading, and its dependent variable, tower diameter; 2) air-to-water ratio; 3) packing shape and size and, thus, packing surface area; and 4) packing height. A variety of combinations of these parameters will be able to achieve the effluent Performance Standards. Thus, the economics of system operation will be a determining factor in selecting the design parameters for the Phase II treatment system.

A comparison of operating conditions between the Phase I and Phase II treatment systems will be conducted to develop the final design criteria. Evaluation of anticipated influent groundwater constituent concentrations (especially methylene chloride) and flow rates will be made; results of these evaluations are required prior to selection of design parameters. The effect of wintertime air temperatures on removal efficiency will also be evaluated, as colder air temperatures than those in the pilot test are anticipated; vertical temperature profiles for the air stripping tower will be estimated, taking into account heat losses in the system. A design safety factor will be developed based on estimated consistency of the concentrations of Constituents of Concern in the groundwater and analysis of the sensitivity of system performance (using design simulations) to changes in operating conditions.

Based on the final configuration of the interception/extraction systems, an analysis of the use of multiple towers will be conducted. This is of particular importance in that the operating requirements (tower height, air flow rate) for removal of methylene chloride are significantly greater than for other constituents, and methylene chloride may not be present in certain areas.

The two packing materials used in the Treatability Study will be compared to other available packings, based on published mass transfer capability, flooding characteristics, gas pressure drop, cost, and operating experience. As available surface area and mass transfer capability are critical to achieving effluent performance standards, these factors will be heavily weighted. Design simulations will be conducted for alternative packings to evaluate their effectiveness.

Operating costs will play a key role in this analysis, due to the long operating life predicted for the Phase II system. Once the packing is selected, analysis of the most economical selection of hydraulic loading and air-to-water ratio will be made by estimating capital and operating cost based on tower size, required hydraulic pumping energy, and air blower energy. A sensitivity analysis will be performed on energy costs. Present worth will be compared for various combinations of the variables to identify the optimal combination.

<u>The operating and performance data developed during the pilot testing, as well as the estimated influent concentrations and effluent water quality criteria, will be used to develop baseline performance criteria for the air stripping system. This baseline performance criteria will be used to prepare a performance specification for the Phase II treatment system (which will be used for system procurement), and will be used to compare manufacturer-proposed equipment to the performance of the pilot Treatment System. The acceptability of the manufacturer-</u>

4.4.6.3 Chemical Scale Control

Based on the characteristics of the groundwater and the Treatability Study results, control of scale will be required for the Phase II treatment system. It should be noted that a significant cost and operational impact can result, depending on the selected method of scale control. Preliminarily identified options for scale control include: 1) Use of phosphate complexing agents to sequester the calcium and prevent formation of calcium carbonate, 2) Addition of acid to the groundwater to offset the pH increase due to carbon dioxide transfer to the water, 3) Periodic acid washing of the packing to remove scale build up, and 4) Use of water softening processes such as ion exchange or reverse osmosis to remove the hardness, consequently limiting calcium available for scaling. Analysis of the cost and operational considerations of scale control alternatives will be conducted, and the preferred method of control will be included in the design.

TABLE ER-4.1

SUMMARY OF ESTIMATED AQUIFER PARAMETERS

	Estimated Aquifer Parameters					
	Transmissivity (ft^2/day)		Hydraulic Conductivity (ft/day)		Storage Coefficient	
	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Storativity	Specific Yield
Upper Sand/Gravel						
CP-S1 APT	10,000	12,000	530	640	–	0.20
RI Estimate[(a)]	600	5,500	90	690	–	–
Lower Sand/Gravel Aquifer						
CP-E1 APT	10,000	14,000	100	140	–	0.18
CP-W1 APT	30,000	40,000	170	230	0.004	–
RI estimate[(a)]	26,000	35,000	60	350	–	–
Basalt Aquifer						
CP-E2 APT	–	25[(b)]	–	0.7[(b)]	0.01	–
RI Estimate[(a)]	–	690	9	26	–	–
Re-Evaluated RI Data[(c)]	–	26[(b)]	–	1.0[(b)]	–	–

(a) From RI Tables 5-2 and 5-3.

(b) Represents estimated average value. Differentiation between upper and lower bounds not warranted because of limited aquifer capacity.

(c) Re-evaluation of hydraulic test recovery data for (b) (6) well, from RI Appendix L.

TABLE ER-4.2

ESTIMATED AQUIFER HORIZONTAL HYDRAULIC GRADIENT AND VELOCITY

Aquifer	Location	Horizontal[(a)] Hydraulic Gradient	Hydraulic[(b)] Conductivity (ft/day)	Average[(c)] Linear Velocity (ft/day)
Upper Sand/Gravel	Near landfill	0.002	530-640	3.5-4.3
Upper Sand/Gravel	South of landfill	0.003	530-640	5.3-6.4
Upper Sand/Gravel	Near CP-S1	0.002	530-640	3.5-4.3
Lower Sand/Gravel	Southwest of landfill	0.0008	170-230	0.5-0.6
Lower Sand/Gravel	Northwest of landfill	0.0008	100-140	0.3-0.4
Basalt	East of landfill	0.06	0.7	0.4

(a) Based on FIgures ER-4.15 and ER-4.20, for Upper and Lower Aquifers, respectively.

(b) From Table ER-4.1.

(c) Assumes an effective porosity of 0.3 for the Upper and Lower Sand/Gravel Aquifers, and 0.1 for the Basalt Aquifer. Values are estimated using the following equation:

$$v = Ki/n_e$$

where v = average linear velocity
K = hydraulic conductivity
i = horizontal hydraulic gradient
n_e = effective porosity

TABLE ER-4.3

PHYSICAL AND CHEMICAL PROPERTIES OF THE CONSTITUENTS OF CONCERN

Constituent	Specific Gravity (unitless)	Solubility[a] (mg/L)	K_{oc}[b] (cm^3/g)	K_d[c] (cm^3/g)	R_d[d] (unitless)	Vapor Pressure (mm)	Henry's Constant ($atm.m^3/mol$)
TCA	1.34	930	130	0.13	1.8	62	0.015
TCE	1.46	1,100	95	0.095	1.6	35	0.010
DCA	1.18	5,500	16	0.016	1.1	115	0.0052
DCE	1.22	2,900	59	0.059	1.4	500	0.018
MC	1.33	21,000	8.7	0.0087	1.1	230	0.0025
PCE	1.62	150	270	0.27	2.7	10	0.014

(a) From Montgomery and Welkom (1990); solubility in water using the average value, or value for temperature closest to 10°C.

(b) Soil Partition Coefficient; average value from Montgomery and Weldom (1990).

(c) Sorption Coefficient; $K_d = (K_{oc})(F_{oc})$, where F_{oc} = fractional organic carbon content of the soil. A value of F_{oc} = 0.001 was assumed for preparation of this table.

(d) Retardation factor; $R_d = Vw/Vc = 1 + BK_d/n_e$, where
Vw = Average linear velocity of groundwater (Lt)
Ve = Average linear velocity of constituent (L/t)
B = Soil bulk density (m/L^3); 1.85 g/cm^3 assumed for preparation of this table.
n_e = effective porosity (unitless); 0.30 assumed for preparation of this table.

TABLE ER-4.4

SUMMARY OF VOLATILE ORGANIC COMPOUNDS DETECTED OTHER THAN THE CONSTITUENTS OF CONCERN

Detected Compound	Number of Detections[a]	Maximum Concentration Detected [b]
Chloroform	12	36[c]
1,2-Dichloroethane	5	3.9
1,2-Dichloropropane	5	3.6
Dichlorodifluoromenthane[d]	1	33
Trichlorofluoromethane	11	48
Vinyl chloride	2	3.1

(a) Indicates number of wells for which compound was detected out of a total of 51 wells sampled during Phase I. Multiple detections from the same well were not included.

(b) All Concentrations in parts per billion (ppb).

(c) Next highest concentration detected is 15 ppb.

(d) Possible laboratory contamination; not detected in duplicate sample.

TABLE ER-4.5

ESTIMATED TCA MIGRATION RATES AND RETARDATION FACTORS

	Upgradient Well		Downgradient Well				
Aquifer	Well Designation	Date of Maximum TCA Concentration	Well Designation	Date of Maximum TCA Concentration	Distance Between Upgradient and Downgradient Wells (ft)	Approximate TCA Migration Rate (ft/day)[(a)]	Apparent Retardation Factor[(b)]
Upper Sand/Gravel Aquifer	Wood	9/84	Henker	2/86	1,275	2.5	2.4
	BN-1	2/83	Volk	6/87	6,750	4.3	1.4
	Volk	6/87	Fredrichsen	8/89	2,325	2.9	2.1
Lower Sand/Gravel Aquifer	CD-2	<4/83[(c)]	CD-42	4/90[(c)]	1,170	$\leq$0.5	$\geq$1.2

(a)

$$\textit{TCA Migration Rate} = \frac{\textit{Distance Between Upgradient and Downgradient Wells}}{\textit{Difference Between Dates of Peak Concentration for Upgradient and Downgradient Wells}}$$

(30 days assumed for each month.)

(b)

$$\textit{Retardation factor} = \frac{\textit{Migration Rate of TCA}}{\textit{Migration Rate of Groundwater}}$$

Groundwater migration rate based on estimated average linear velocities provided in Table ER-4.2; 6.0 ft/day and 0.6 ft/day were used as average linear velocities for the Upper and Lower Sand/Gravel Aquifers, respectively.

(c) Date correspond to first detection of TCA, not the time of maximum concentration.

TABLE ER-4.6

ESTIMATED CONTAMINANT MASS IN GROUND WATER FLOW SYSTEM

	Estimated Mass in Ground Water Flow System[(a)]		Estimated Mass Disposed of at Colbert Landfill[(b)]	
Constituent	Mass (lb)	Volume (gal)	Mass (lb)	Volume (gal)
TCA	40,000	3,600	140,000	13,000
DCE	3,300	320	--[(c)]	--
DCA	660	67	--	--
TCE	1,400	120	--	--
MC	6,800	610	240,000	22,000
PCE	4.1	0.3	--	--

(a) Combined soil and water fractions in saturated zone, see Appendix G for estimation procedures.

(b) Disposed mass, based on RI Table 1-1.

(c) Information not available.

TABLE ER-4.7

SUMMARY OF GENERAL GROUND WATER QUALITY ANALYTICAL RESULTS (a)

Analysis Parameter	Well No.:	CD-21C1	CD-21C3	CD-21C3 (dup)	CD-30A	CD-32A	CD-42C1	CD-42C2	(CD-42C2) (dup)	CD-42C3
	Sample No.:	123	121	122	440	138	107	108	109	110
	Date:	1/31/90	1/31/90	1/31/90	4/2/91	2/27/90	1/18/90	1/18/90	1/18/90	1/18/90
Landfill Leachate Parameters										
Cadmium (dissolved)		0.001 U(c)	0.001 U	--	0.001 U	0.0001 U	0.001 U	0.001	--	0.001 U
Chloride		6	2	2	22	--	3	4	4	2
Sulfate as SO4		19	13	--	--	12	13	16	--	15
Nitrate + Nitrite as N		1.5	0.59	0.56	2.6	0.05 U	2	4.7	4.9	0.05 U
Total Dissolved Solids (TDS)		650	310	290	430	--	270	330	300	260
Total Organic Carbon (TOC)		2	1 U	1.2	2.2	--	1.1	1.3	1.9	2.1
Total Organic Halides (TOX)		2.2	0.02 U	0.02 U	0.4	--	0.02 U	0.02 U	0.02 U	0.02 U
Chemical Oxygen Demand (COD)		10 U	10 U	10 U	10 U	--	10 U	10 U	10 U	10 U
Other Parameters										
Iron (dissolved)		0.016	0.1	--	0.2 J	0.13	0.011 J(d)	0.2 J(d)	--	0.29 J(d)
Manganese (dissolved)		0.29	0.73	--	0.003	0.11	0.12	0.064	--	0.34
Magnesium (dissolved)		39	13	--	1.3	16	19 J	17 J	--	14 J
Calcium (dissolved)		200	46	--	59	46	52	44	--	44
Potassium (dissolved)		64	10 U	--	4.5	4.3	3.5	3.2	--	3.4
Sodium (dissolved)		8.8	21	--	0.01 U	9	7.8	25	--	17
Hardness as CaCO3		740	190	190	370	220	260	210	210	190
pH		7.09	7.84	7.84	7.13	7.64	7.70	7.60	7.60	7.80
Conductivity (umhos/cm)		978	401	401	666	407	440	448	448	398

(a) Parts per million, except where indicated otherwise.
(b) -- = Not analyzed.
(c) U = The analyte was not detected, to the limit of detection indicated.
(d) J = estimated value; the analyte of interest was detected in the method blank associated with the sample, as well as in the sample itself.

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TABLE ER-4.8
TREATABILITY STUDY ANALYTICAL RESULTS(a)

Trial No.	Location	Date Collected	Air Flow (CFM)(b)	Hydraulic Load (GPM)	TCA	Methylene Chloride (c)	TCE	DCE	DCA	PCE	Hardness (mg/L)	Alkalinity (mg/L)	TDS (mg/L)	pH	Conductivity (umhos/cm)	Temperature (C) Air(d)	Temperature (C) Water
1	Influent #3	1/29/91	2010	203	2600	3000	46	340	52	1.9	NT(e)	NT	NT	6.9	1226	-1	11.3
	Influent #2				2600	3100	57	360	57	2.2	NT	NT	NT	6.9	1218		11.2
	Influent #1				2600	3200	76	350	100	2.3	820	800	830	6.8	1228		10.4
	Influent 40' (f)				2000	2700	48	210	50	1.7	800	800	830	8.1	1168		10.6
	Influent 30'				340	750	10	26	13	0.30 U(g)	790	800	590	8.3	1166		9.2
	Influent 20'				30	130	1.3	2.2	2.0	0.30 U	770	780	570	8.2	1180		10.3
	Influent 10'				6.9	38	1.2 U	0.13 U	0.70 U	0.30 U	680	690	550	8.0	1192		10.8
	Effluent #1				3.2	19	1.2 U	0.13 U	0.70 U	0.30 U	660	670	540	7.1	1209	11	11.2
	Effluent #2				3.5	23	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.2	1168		10.8
	Effluent #3				3.9	24	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.1	1170		10.9
	Trip Blank				0.3 U	<4	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
2	Influent #3	1/30/91	2160	200	2500	2700	42	290	55	2.1	NT	NT	NT	7.0	1226	-2	11.2
	Influent #2				2600	2700	44	290	57	2.2	NT	NT	NT	7.0	1226		11.3
	Influent #1				2600	2700	49	300	56	2.2	800	800	580	6.9	1168		11.1
	Influent 40'				2000	2400	39	190	50	1.6	780	800	830	7.2	1226		10.8
	Influent 30'				470	820 B	8.5	26	13	0.30 U	810	790	590	8.0	1194		10.9
	Influent 20'				29	66	1.2 U	2.1	2.1	0.30 U	780	790	570	8.3	1100		10.8
	Influent 10'				6.2	15 B	1.2 U	0.13 U	0.7 U	0.30 U	680	720	530	8.3	1085		9.4
	Effluent #1				3.6	7.6 B	1.2 U	0.13 U	0.7 U	0.30 U	740	730	500	8.2	1215	11	10.5
	Effluent #2				2.2	8.7	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.2	1168		10.6
	Effluent #3				3.6	10	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.2	1170		10.7
3	Influent #3	1/30/91	1810	203	2700	3000	51	300	56	2.2	NT	NT	NT	7.1	1223	-2	10.8
	Influent #2				2600	2800	52	280	58	2.1	NT	NT	NT	6.9	1225		10.8
	Influent #1				2500	2100	41	240	56	2.2	800	800	840	6.8	1240		8.2
	Influent 40'				2000	2500	46	160	49	1.5	800	800	820	7.2	1221		11.8
	Influent 30'				550	1000	10	30	14	0.30 U	890	790	580	8.0	1201		10.6
	Influent 20'				39	88	1.4	2.4	2.6	0.30 U	790	800	570	8.2	1187		10.2
	Influent 10'				7.4	24	1.2 U	0.13 U	0.70 U	0.30 U	840	750	460	8.2	1188		8.9
	Effluent #1				5.2	18	1.2 U	0.13 U	0.70 U	0.30 U	760	760	560	8.2	1181	11	8.6
	Effluent #2				4.7	19 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.2	1168		11.8
	Effluent #3				5.4	20	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.2	1166		11.8
	Trip Blank				0.3 U	2.5 U	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
4	Influent #3	3/11/91	1390	151	2200	2100	55	340	56	2.4	NT	NT	NT	6.73	1206	8	11.0
	Influent #2				2400	2300	58	390	59	2.6	NT	NT	NT	6.70	1208		11.6
	Influent #1				2300	2200	52	370	56	2.6	1000	780	800	6.63	1220		11.5
	Influent 40'				1800	1900	52	250	51	2.0	820	790	820	6.94	1201		10.4
	Influent 30'				350	520	8.1	18	10	0.30 U	860	720	810	7.92	1175		9.7
	Influent 20'				49	160	2.4	4.5	2.4	0.30 U	810	660	820	8.03	1170		10.7
	Influent 10'				11	54	1.2 U	0.13 U	0.70 U	0.30 U	790	590	800	8.07	1167		10.7
	Effluent #1				4.1	21	1.2 U	0.13 U	0.70 U	0.30 U	740	610	800	8.11	1175	11	10.7
	Effluent #2				4.9	24	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.14	1167		11.0
	Effluent #3				6.3	24	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.15	1173		11.2
	Trip Blank				0.3 U	4.1 U	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT

TABLE ER-4.8

TREATABILITY STUDY ANALYTICAL RESULTS(a)

Trial No.	Location	Date Collected	Air Flow (CFM)(b)	Hydraulic Load (GPM)	TCA	Methylene Chloride (c)	TCE	DCE	DCA	PCE	Hardness (mg/L)	Alkalinity (mg/L)	TDS (mg/L)	pH	Conductivity (umhos/cm)	Temperature (C) Air(d)	Temperature (C) Water
5	Influent #3	3/12/91	1500	151	1800	2100	44	260	55	2.3	NT	NT	NT	6.46	1268	5	11.8
	Influent #2				2200	2300	42	350	55	2.4	NT	NT	NT	6.10	1231		11.7
	Influent #1				2200	2300	42	350	52	2.5	800	790	830	6.65	1221		12.0
	Influent #1 (Dup)(h)				2200	2300	41	350	52	2.4	820	800	820	6.66	1215		11.9
	Influent 40'				1700	1900	40	230	49	1.9	820	780	830	6.41	1258		11.5
	Influent 30'				320	630	6.8	21	9.7	0.30 U	770	740	800	6.22	1216		10.7
	Influent 20'				43	140	1.6	3.4	2.9	0.30 U	710	650	720	7.65	1173		11.3
	Influent 10'				8.5	43	1.2 U	0.13 U	0.70 U	0.30 U	700	650	700	8.03	1163		11.3
	Effluent #1				4.9	21	1.2 U	0.13 U	0.70 U	0.30 U	660	650	700	8.09	1169	11	11.3
	Effluent #2				4.7	18	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	7.61	1202		11.3
	Effluent #3				4.3	18	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	7.97	1226		11.2
6	Influent #3	3/12/91	1650	151	2300	2300	39	1400 (i)	55	2.2	NT	NT	NT	NT	NT	5	11.7
	Influent #2				2200	2300	41	340	56	2.4	NT	NT	NT	NT	NT		11.8
	Influent #1				1100 (i)	1900	44	73 (i)	57	2.2	820	790	830	6.9	1300		11.8
	Influent 40'				1800	2000	36	230	48	0.30 U	800	770	820	7.2	1300		11.4
	Influent 30'				140	260	5.4	18	8.0	0.30 U	760	740	790	8.0	1200		11.3
	Influent 20'				39	110	1.5	3.2	2.5	0.30 U	720	690	740	8.0	1100		11.3
	Influent 10'				7.6	28	1.2 U	0.13 U	0.70 U	0.30 U	680	650	690	7.9	1100		11.3
	Effluent #1				3.4	12	1.2 U	0.13 U	0.70 U	0.30 U	730	650	710	7.9	1100		11.3
	Effluent #1 (Dup)				3.5	11	1.2 U	0.13 U	0.70 U	0.30 U	670	640	700	8.0	1100	11	11.3
	Effluent #2				3.5	10	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		11.3
	Effluent #3				3.7	10	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		11.3
	Trip Blank				0.3 U	<3.4	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
7	Transfer Blank	3/13/91	1650	150	0.3 UJ(j)	2.5 UJ	1.2 UJ	0.13 UJ	0.70 UJ	0.30 UJ	NT	NT	NT	NT	NT		NT
	Influent #3				2200	2100	74	410	60	2.5	NT	NT	NT	6.74	1178	9	12.0
	Influent #2				1900	1800	84	320	64	2.8	NT	NT	NT	6.72	1183		11.7
	Influent #1				2100	1900	130 (i)	370	65	2.7	780	730	800	6.67	1172		12.0
	Influent 40'				1800	1800	75	300	55	2.0	660	750	790	6.94	1171		11.6
	Influent 30'				170	270	11	22	8.7	0.30 U	850	710	760	7.92	1141		11.4
	Influent 20'				38	99	3.0	3.0	2.5	0.30 U	720	680	720	8.02	1139		11.8
	Influent 10'				8.0	27	1.2 U	0.13 U	0.70 U	0.30 U	670	630	690	8.04	1131		11.8
	Effluent #1				2.4	8.5	1.2 U	0.13 U	0.70 U	0.30 U	390	650	680	8.03	1228	12	11.5
	Effluent #2				3.9	9.3	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.06	1136		11.2
	Effluent #3				4.0	9.1	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.06	1132		11.5
	Trip Blank				0.3 U	2.5 U	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
8	Influent #3	3/14/91	1330	148	2300	2300	140	240	73	2.1	NT	NT	NT	6.74	1191	11	11.7
	Influent #2				2600	2500	71	290	62	1.8	NT	NT	NT	6.75	1183		11.7
	Influent #1				2300	2300	70	220	66	1.8	820	750	760	6.69	1199		11.6
	Influent 40'				1900	2000	78	150	60	1.5	810	750	790	6.97	1179		11.6
	Influent 30'				150	340	9.2	14	8.2	0.30 U	790	720	750	7.92	1181		11.6
	Influent 20'				37	130	2.6	2.2	2.7	0.30 U	730	680	680	8.04	1150		11.4
	Influent 10'				7.0	38	1.2 U	0.13 U	0.70 U	0.30 U	760	650	670	8.05	1153		11.5
	Effluent #1				2.6	16 B	1.2 U	0.13 U	0.70 U	0.30 U	880	630	630	8.06	1159	12	11.9
	Effluent #2				1.9	13 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.08	1134		11.5
	Effluent #3				3.5	18	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.08	1140		11.3
	Trip Blank				0.3 UJ	4.4 UJ	1.2 UJ	0.13 UJ	0.70 UJ	0.30 UJ	NT	NT	NT	NT	NT		NT

TABLE ER-4.8

TREATABILITY STUDY ANALYTICAL RESULTS(a)

Trial No.	Location	Date Collected	Air Flow (CFM)(b)	Hydraulic Load (GPM)	TCA	Methylene Chloride (c)	TCE	DCE	DCA	PCE	Hardness (mg/L)	Alkalinity (mg/L)	TDS (mg/L)	pH	Conductivity (umhos/cm)	Temperature (C) Air(d)	Temperature (C) Water
9	Transfer Blank	3/18/91	1360	153	0.3 U	2.5 U	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
	Influent #3				2100	2300	80	280	69	2.0	NT	NT	NT	6.93	1204	16	11.9
	Influent #2				2300	2400	71	310	65	2.1	NT	NT	NT	6.93	1201		11.9
	Influent #1				1800	2000	1000 (i)	200	73	2.2	820	740	800	6.67	1198		11.5
	Influent 40'				1600	2000	62	160	55	1.4	770	750	800	7.13	1194		11.7
	Influent 40' (Dup)				NT	NT	NT	NT	NT	NT	820	760	810	7.13	1194		11.7
	Influent 30'				240	550	14	28	12	0.30 U	770	710	760	7.99	1172		11.6
	Influent 20'				31	92	1.2 U	1.5	1.6	0.30 U	690	660	720	8.13	1164		11.1
	Influent 10'				4.9	17	1.2 U	0.13 U	0.70 U	0.30 U	670	640	680	8.16	1160		11.8
	Effluent #1				0.3 U	<4.2	1.2 U	0.13 U	0.70 U	0.30 U	650	610	670	8.14	1167	12	12.3
	Effluent #1 (Dup)				NT	NT	NT	NT	NT	NT	650	620	680	8.14	1167		12.3
	Effluent #2				0.3 U	<4.0	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.28	1166		12.1
	Effluent #2 (Dup)				0.3 U	<7.6	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.28	1166		12.1
	Effluent #3				0.3 U	<5.4	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.28	1168		12.4
10	Transfer Blank	3/18/91	1360	154	0.3 U	2.5 U	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
	Influent #3				2800	3000	58	310	76	2.0	NT	NT	NT	6.67	1226	17	11.8
	Influent #2				2800	3100	53	310	68	1.8	NT	NT	NT	6.95	1229		11.9
	Influent #1				2200	2400	49	280	66	1.9	830	780	830	6.56	1239		12.0
	Influent #1 (Dup)				2100	2400	45	290	61	1.7	840	790	820	6.56	1239		12.0
	Influent 40'				2100	2700	46	180	66	1.3	840	780	830	6.89	1224		11.6
	Influent 30'				300	700	10	27	16	0.30 U	800	760	790	7.79	1197		11.5
	Influent 20'				30	120	1.2 U	0.13 U	1.5	0.30 U	780	680	720	7.93	1193		11.9
	Influent 20' (Dup)				29	120	1.2 U	0.13 U	1.5	0.30 U	760	680	710	7.93	1193		11.9
	Influent 10'				4.4	23	1.2 U	0.13 U	0.70 U	0.30 U	680	650	690	8.02	1182		12.0
	Effluent #1				0.3 U	<5.2	1.2 U	0.13 U	0.70 U	0.30 U	660	630	670	8.03	1190		11.9
	Effluent #2				0.3 U	6.7	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.05	1183	12	11.8
	Effluent #3				0.3 U	5.5	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.07	1182		11.7
	Trip Blank				0.3 U	2.4	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
	Transfer Blank				0.3 U	1.5 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
11	Influent #3	3/19/91	1520	155	2900	2100	50	210	65	1.8	NT	NT	NT	6.69	1231	11	12.0
	Influent #2				3100	2300	46	270	62	1.2	NT	NT	NT	6.69	1235		12.0
	Influent #1				2600	2600	45	220	57	1.0	840	780	840	6.64	1237		12.3
	Influent 40'				2200	1700	37	140	64	1.0	830	780	820	6.94	1227		12.0
	Influent 40 ' (Dup)				NT	NT	NT	NT	NT	NT	880	790	810	6.94	1227		12.0
	Influent 30'				230	180	8.0	22	12	0.30 U	830	770	790	7.86	1201		12.0
	Influent 20'				20	85	1.2 U	0.13 U	0.96	0.30 U	760	700	750	8.03	1193		12.0
	Influent 10'				2.0	20	1.2 U	0.13 U	0.70 U	0.30 U	680	660	700	8.08	1183		11.9
	Effluent #1				0.3 U	<6.0	1.2 U	0.13 U	0.70 U	0.30 U	700	670	710	8.12	1184	12	12.2
	Effluent #2				0.3 U	5.5	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.11	1180		11.7
	Effluent #3				0.3 U	3.6 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.10	1177		11.8

TABLE ER-4.8

TREATABILITY STUDY ANALYTICAL RESULTS(a)

Trial No.	Location	Date Collected	Air Flow (CFM)(b)	Hydraulic Load (GPM)	TCA	Methylene Chloride (c)	TCE	DCE	DCA	PCE	Hardness (mg/L)	Alkalinity (mg/L)	TDS (mg/L)	pH	Conductivity (umhos/cm)	Temperature (C) Air(d)	Temperature (C) Water
12	Transfer Blank	3/19/91	1620	155	0.3 U	3.0 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	6.65	1230	14	12.0
	Influent #3				2100	2300	44	100 (i)	60	1.4	NT	NT	NT	6.67	1225		12.0
	Influent #2				2300	2000	51	170	64	1.8	NT	NT	NT	NT	NT		NT
	Influent #1				2800	2100	51	230	67	1.8	830	790	820	6.65	1233		12.1
	Influent 40'				1600	180	36	69	63	1.0	880	790	810	6.90	1221		11.9
	Influent 30'				170	140	6.5	17	10	0.30 U	780	750	780	7.81	1196		11.8
	Influent 30' (Dup)				NT	NT	NT	NT	NT	NT	780	740	770	7.81	1196		11.8
	Influent 20'				17	46	1.2 U	0.13 U	0.70 U	0.30 U	720	700	750	7.97	1189		12.0
	Influent 10'				3.6	13	1.2 U	0.13 U	0.70 U	0.30 U	700	650	680	8.01	1181		11.9
	Influent 10' (Dup)				3.4	14	1.2 U	0.13 U	0.70 U	0.30 U	680	660	680	8.01	1181		11.9
	Effluent #1				0.3 U	3.7 B	1.2 U	0.13 U	0.70 U	0.30 U	710	670	700	8.06	1177	12	11.8
	Effluent #2				0.3 U	3.8	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.02	1175		11.8
	Effluent #3				0.3 U	2.3 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.02	1167		11.7
	Trip Blank				0.3 U	2.8	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
13	Influent #3	3/20/91	1620	150	2300	1600	71	160	61	1.7	NT	NT	NT	6.74	1192	9	11.9
	Influent #2				2400	<400 (i)	77	160	62	1.8	NT	NT	NT	6.72	1183		11.7
	Influent #1				2300	1500	77	150	66	1.8	780	760	810	6.67	1198		11.3
	Influent '40				1700	2000	58	170	50	1.3	780	750	810	6.97	1186		12.0
	Influent '30				2000	300	11	25	10	0.30 U	740	730	780	7.88	1157		11.6
	Influent '30 (Dup)				2200	250	11	25	11	0.30 U	700	730	780	7.87	1157		11.5
	Influent '20				23	56	1.2 U	0.70	1.5	0.30 U	640	700	740	8.03	1145		11.6
	Influent '10				3.2	9.2 B	1.2 U	0.13 U	0.70 U	0.30 U	630	660	710	8.06	1134		11.5
	Effluent #1				0.3 U	2.5 U	1.2 U	0.13 U	0.70 U	0.30 U	630	670	710	8.08	1142	11	11.3
	Effluent #2				0.3 U	2.5 U	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.10	1128		11.6
	Effluent #3				0.3 U	2.8	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.06	1140		11.7
	Transfer Blank				0.3 U	2.5 U	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
14	Influent #3	3/20/91	1790	201	2300	68 (i)	44	230	69	1.4	NT	NT	NT	6.70	1224	11	12.0
	Influent #2				2200	2400	43	200	60	1.2	NT	NT	NT	6.69	1227		11.9
	Influent #1				2200	2400	43	190	60	1.1	810	780	830	6.66	1221		12.1
	Influent '40				1500	2200	33	100	50	0.87	810	780	850	6.96	1217		11.8
	Influent '30				150	170	7.2	19	11	0.30 U	720	760	780	7.86	1186		11.8
	Influent '20				19	66	1.2 U	0.64	0.89	0.30 U	670	720	750	8.00	1183		12.3
	Influent '10				2.3	15	1.2 U	0.13 U	0.70 U	0.30 U	650	680	730	7.99	1175		11.8
	Effluent #1				0.3 U	3.2 B	1.2 U	0.13 U	0.70 U	0.30 U	640	680	710	8.03	1168	12	11.9
	Effluent #2				0.3 U	5.0	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.04	1168		11.6
	Effluent #3				0.3 U	5.5	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.04	1164		11.7
	Trip Blank				0.3 U	2.2 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
15	Influent #3	3/21/91	1960	201	1600	1300	44	140	62	1.3	NT	NT	NT	6.70	1239	8	12.2
	Influent #2				1500	1000	46	130	65	1.6	NT	NT	NT	6.69	1234		12.2
	Influent #1				2700	2200	42	220	65	1.1	810	790	830	6.68	1220		12.1
	Influent 40'				1200	1100	35	88	54	1.1	800	780	830	6.96	1224		12.2
	Influent 30'				230	180	6.1	14	11	0.30 U	700	740	740	7.90	1196		12.1
	Influent 20'				38	77	1.2 U	4.3	1.7	0.30 U	670	700	730	8.02	1184		12.0
	Influent 10'				1.5	13	1.2 U	0.13 U	0.70 U	0.30 U	660	710	730	8.05	1173		12.1
	Influent 10' (Dup)				1.9	14	1.2 U	0.13 U	0.70 U	0.30 U	650	710	720	8.05	1173		12.1
	Effluent #1				0.3 U	3.2	1.2 U	0.13 U	0.70 U	0.30 U	640	670	680	8.09	1166	12	11.8
	Effluent #2				0.3 U	9.0	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.08	1178		11.9
	Effluent #3				0.3 U	7.5	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.05	1183		12.2
	Transfer Blank				0.3 U	2.5 U	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT

TABLE ER-4.8

TREATABILITY STUDY ANALYTICAL RESULTS(a)

Trial No.	Location	Date Collected	Air Flow (CFM)(b)	Hydraulic Load (GPM)	TCA	Methylene Chloride (c)	TCE	DCE	DCA	PCE	Hardness (mg/L)	Alkalinity (mg/L)	TDS (mg/L)	pH	Conductivity (umhos/cm)	Temperature (C)	
																Air(d)	Water
16	Influent #3	3/21/91	2170	201	2400	2000	46	250	57	1.7	NT	NT	NT	6.67	1232	15	12.0
	Influent #2				2600	2000	42	260	53	1.6	NT	NT	NT	6.67	1230		12.0
	Influent #1				1700	1200	43	100 (i)	63	1.3	800	780	830	6.63	1231		12.9
	Influent 40'				1200	1200	36	190	46	1.3	790	790	830	6.95	1228		12.1
	Influent 30'				240	420	6.3	22	9.4	0.30 U	730	750	770	7.87	1194		11.9
	Influent 20'				22	66	1.2 U	0.13 U	1.0	0.30 U	660	690	710	8.04	1183		12.0
	Influent 10'				3.4	11	1.2 U	0.13 U	0.70 U	0.30 U	650	700	710	7.94	1184		12.1
	Effluent #1				0.3 U	<3.5	1.2 U	0.13 U	0.70 U	0.30 U	650	690	710	8.02	1171	12	11.9
	Effluent #2				0.3 U	<5.6	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.04	1173		11.8
	Effluent #3				2.2	6	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.03	1169		11.8
	Trip Blank				0.3 U	2.5	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
17	Influent #1	1/31/91	2400	200	2500	2600	40	290	54	2	NT	NT	NT	6.78	1237		11.3
	Influent #2				2400	2600	44	290	56	0.30 U	NT	NT	NT	6.77	1243		11.3
	Influent #3				2500	2700	44	300	56	1.9	NT	NT	NT	6.76	1240		11.5
	Influent #4				2500	2700	44	270	56	2	NT	NT	NT	6.77	1243		11.5
	Influent #5				2400	2700	43	260	55	2	NT	NT	NT	6.81	1245		11.6
	Influent #6				2600	2600	36	280	58	2	NT	NT	NT	6.81	1233		11.5
	Effluent #1				2.4	5 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.01	1186		10.9
	Effluent #2				1.7	5.2 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.05	1183		10.7
	Effluent #3				2.5	5.4 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.07	1178		10.7
	Effluent #4				2.7	6 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.08	1170		10.7
	Effluent #5				2.1	6.2	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.08	1186		10.7
	Effluent #6				2.5	7.2	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	8.09	1188		10.7
	Transfer Blank				0.3 U	2.2 B	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT
	Trip Blank				1.6	2.5 U	1.2 U	0.13 U	0.70 U	0.30 U	NT	NT	NT	NT	NT		NT

(a) Units are ug/L unless otherwise indicated.
(b) Air flow standardized to 32 Deg. F, 1 atm. Influent air temperature is air entering at bottom of tower, effluent air temperature is air exiting top of tower.
(c) Detected levels of methylene chloride have been corrected for blank contamination. The results have been qualified in the following manner:
< = Sample result is less than or equal to 2 times the associated laboratory blank concentration.
B = Sample result is greater than 2 times, but less than 5 times the associated laboratory blank concentration.
If the sample result is greater than 5 times the associated laboratory blank concentration, no data flags are assigned.
(d) Average of air temperatures recorded.
(e) NT = Not tested.
(f) Influent sample taken from sampling port 40' above the base of the tower.
(g) U = The analyte of interest was not detected, to the limit of detection indicated.
(h) Duplicate sample.
(i) Datum eliminated from data analysis.
(j) J = At least one surrogatge spike compound was reported outside the control limits established in the QAPjP.

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TABLE ER-4.9
TREATABILITY STUDY DESIGN VARIABLES AND TEST RESULTS

Trial Number	Nominal Packing Diameter (inches)	Methylene Chloride Concentration (a) Influent (ug/l)	Methylene Chloride Concentration (a) Effluent (ug/l)	Air Flow Rate (CFM)(b)	Hydraulic Loading Rate (gpm)	Air/Water Ratio (Vol/Vol)	Calculated (c) Mass Transfer Constant (Kla) (1/hr)	Measured (d) Mass Transfer Constant (Kla) (1/hr)	Removal Efficiency (%)
1	3.5	3100	22	2010	203	74.2	17.19	25.84	99.29
2	3.5	2700	8.8	2160	200	80.8	17.74	28.99	99.67
3	3.5	2633	19	1810	203	66.8	16.24	26.59	99.30
4	3.5	2200	23	1390	151	68.7	12.54	18.11	98.96
5	3.5	2233	19	1500	151	74.3	13.09	18.54	99.15
6	3.5	2167	10.8	1650	151	81.5	13.76	20.25	99.51
7	3.5	1933	9	1650	150	82.1	13.73	20.32	99.53
8	3.5	2367	15.7	1330	148	67.3	12.17	19.77	99.34
9	2	2230	5.3	1360	153	66.5	29.40	25.63	99.76
10	2	2725	5.8	1360	154	66.1	29.50	25.82	99.79
11	2	2333	5	1520	155	73.2	30.49	24.91	99.78
12	2	2133	3.3	1620	155	78.4	31.04	25.82	99.85
13	2	1550	2.6	1620	154	78.9	30.92	25.24	99.83
14	2	2400	4.6	1790	201	66.7	37.04	33.91	99.81
15	2	1500	6.6	1960	201	73.0	37.91	28.36	99.56
16	2	1733	5	2170	201	80.6	38.86	29.76	99.71

(a) Average of Concentrations Measured
(b) Air flow standardized to 32 deg. F, 1 atm.
(c) Using uncalibrated VOLSTRIP model.
(d) Using Treatability Study data.

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TABLE ER-4.10
HYDRAULIC LOADING DESIGN VARIABLE ANALYSIS

Trial Number	Nominal Packing Diameter (inches)	Methylene Chloride Concentration (a) Influent (ug/l)	Methylene Chloride Concentration (a) Effluent (ug/l)	Air Flow Rate (cfm)(b)	Hydraulic Loading Rate (gpm)	Air/Water Ratio (Vol/Vol)	Calculated (c) Mass Transfer Constant (Kla) (1/hr)	Measured (d) Mass Transfer Constant (Kla) (1/hr)	Removal Efficiency (%)	Average Removal Efficiency for Group (%)	Average Removal Efficiency for 3.5 In. Packing In Group	Average Removal Efficiency for 2 In. Packing In Group
Group 1 - Nominal 200 gpm Hydraulic Loading												
1	3.5	3100	22	2010	203	74.2	17.19	25.84	99.29			
2	3.5	2700	8.8	2160	200	80.8	17.74	28.99	99.67			
3	3.5	2633	19	1810	203	66.8	16.24	26.59	99.3			
14	2	2400	4.6	1790	201	66.7	37.04	33.91	99.81			
15	2	1500	6.6	1960	201	73	37.91	28.36	99.56			
16	2	1733	5	2170	201	80.6	38.86	29.76	99.71	99.56	99.42	99.69
Group 2 - Nominal 150 gpm Hydraulic Loading												
4	3.5	2200	23	1390	151	68.7	12.54	18.11	98.96			
5	3.5	2233	19	1500	151	74.3	13.09	18.54	99.15			
6	3.5	2167	10.8	1650	151	81.5	13.76	20.25	99.51			
7	3.5	1933	9	1650	150	82.1	13.73	20.32	99.53			
8	3.5	2367	15.7	1330	148	67.3	12.17	19.77	99.34			
9	2	2230	5.3	1360	153	66.5	29.4	25.63	99.76			
10	2	2725	5.8	1360	154	66.1	29.5	25.82	99.79			
11	2	2333	5	1520	155	73.2	30.49	24.91	99.78			
12	2	2133	3.3	1620	155	78.4	31.04	25.82	99.85			
13	2	1550	2.6	1620	154	78.9	30.92	25.24	99.83	99.55	99.30	99.80

(a) Average of Concentrations Measured
(b) Air flow standardized to 32 deg. F, 1 atm.
(c) Using uncalibrated VOLSTRIP model.
(d) Using Treatability Study data.

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TABLE ER-4.11

AIR-TO-WATER RATIO DESIGN VARIABLE ANALYSIS

Trial Number	Nominal Packing Diameter (inches)	Methylene Chloride Concentration (a) Influent (ug/l)	Methylene Chloride Concentration (a) Effluent (ug/l)	Air Flow Rate (CFM)(b)	Hydraulic Loading Rate (gpm)	Air/Water Ratio (Vol/Vol)	Calculated (c) Mass Transfer Constant (Kla) (1/hr)	Measured (d) Mass Transfer Constant (Kla) (1/hr)	Removal Efficiency (%)	Average Removal Efficiency for Group (%)	Average Removal Efficiency for 3.5 in. Packing in Group (%)	Average Removal Efficiency for 2 in. Packing in Group (%)
Group 1 - All Air/Water Ratios 80 or Greater												
2	3.5	2700	8.8	2160	200	80.8	17.74	28.99	99.67			
6	3.5	2167	10.8	1650	151	81.5	13.76	20.25	99.51			
7	3.5	1933	9	1650	150	82.1	13.73	20.32	99.53			
16	2	1733	5	2170	201	80.6	38.86	29.76	99.71	99.61	99.57	99.71
Group 2 - All Air/Water Ratios 70 to 80												
1	3.5	3100	22	2010	203	74.2	17.19	25.84	99.29			
5	3.5	2233	19	1500	151	74.3	13.09	18.54	99.15			
11	2	2333	5	1520	155	73.2	30.49	24.91	99.78			
12	2	2133	3.3	1620	155	78.4	31.04	25.82	99.85			
13	2	1550	2.6	1620	154	78.9	30.92	25.24	99.83			
15	2	1500	6.6	1960	201	73	37.91	28.36	99.56	99.58	99.22	99.76
Group 3 - All Air/Water Ratios 60 to 70												
3	3.5	2633	19	1810	203	66.8	16.24	26.59	99.3			
4	3.5	2200	23	1390	151	68.7	12.54	18.11	98.96			
8	3.5	2367	15.7	1330	148	67.3	12.17	19.77	99.34			
9	2	2230	5.3	1360	153	66.5	29.4	25.63	99.76			
10	2	2725	5.8	1360	154	66.1	29.5	25.82	99.79			
14	2	2400	4.6	1790	201	66.7	37.04	33.91	99.81	99.49	99.20	99.79

(a) Average of Concentrations Measured

(b) Air flow standardized to 32 deg. F, 1 atm.

(c) Using uncalibrated VOLSTRIP model.

(d) Using Treatability Study data.

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TABLE ER-4.12
COMPARISON OF PERFORMANCE OF PACKING DIAMETERS

Trial Number	Nominal Packing Diameter (inches)	Methylene Chloride Concentration Influent (ug/l)	Methylene Chloride Concentration Effluent (ug/l)	Air Flow Rate (CFM)(a)	Hydraulic Loading Rate (gpm)	Air/Water Ratio (Vol/Vol)	Calculated (b) Mass Transfer Constant (Kla) (1/hr)	Measured (c) Mass Transfer Constant (Kla) (1/hr)	Removal Efficiency (%)	Average Removal Efficiency for Group (%)
Group 1 - All 3.5 Inch Packing										
1	3.5	3100	22	2010	203	74.2	17.19	25.84	99.29	
2	3.5	2700	8.8	2160	200	80.8	17.74	28.99	99.67	
3	3.5	2633	19	1810	203	66.8	16.24	26.59	99.3	
4	3.5	2200	23	1390	151	68.7	12.54	18.11	98.96	
5	3.5	2233	19	1500	151	74.3	13.09	18.54	99.15	
6	3.5	2167	10.7	1650	151	81.5	13.76	20.25	99.51	
7	3.5	1933	9	1650	150	82.1	13.73	20.32	99.53	
8	3.5	2367	15.7	1330	148	67.3	12.17	19.77	99.34	99.34
Group 2 - All 2 Inch Packing										
9	2	2230	5.3	1360	153	66.5	29.4	25.63	99.76	
10	2	2730	5.8	1360	154	66.1	29.5	25.82	99.79	
11	2	2330	5	1520	155	73.2	30.49	24.91	99.78	
12	2	2130	3.3	1620	155	78.4	31.04	25.82	99.85	
13	2	1550	2.6	1620	154	78.9	30.92	25.24	99.83	
14	2	2400	4.6	1790	201	66.7	37.04	33.91	99.81	
15	2	1500	6.6	1960	201	73	37.91	28.36	99.56	
16	2	1733	5	2170	201	80.6	38.86	29.76	99.71	99.76

(a) Air flow standardized to 32 deg. F, 1 atm.
(b) Using uncalibrated VOLSTRIP model.
(c) Using Treatability Study data.

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TABLE ER-4.13

MAXIMUM AND AVERAGE MASS FLUX ESTIMATES

Compound	Estimated Design Influent Concentration[a] (ppb)	Estimated Maximum Mass Flux[b] (g/sec)	Estimated Constitutent Mass[c] (kg)	Estimated Average Mass Flux[d] (g/s)
1,1,1-TCA	1,700	2.09×10^{-1}	18,000	1.90×10^{-2}
1,1-DCE	90	1.08×10^{-2}	1,500	1.59×10^{-3}
1,1-DCA	110	1.34×10^{-2}	300	3.17×10^{-4}
TCE	25	2.95×10^{-3}	650	6.87×10^{-4}
PCE	4	4.29×10^{-4}	1.9	2.01×10^{-6}
Methylene Chloride	560	6.76×10^{-2}	3,100	3.28×10^{-3}

(a) Based on Appendix F of the FS.

(b) Based on estimated design influent concentration, and a hydraulic loading of 1,900 gpm.

(c) Combined soil and water fractions in the saturated zone, based on Phase I data and domestic well data.

(d) Based on estimated constituent mass, a hydraulic loading of 1,900 gpm, and a 30-year operational period.

TABLE ER-4.14

HEALTH RISK ASSESSMENT SUMMARY INFORMATION

Constituent	Average Annual Concentration Based on Maximum Mass Flux[a] ($\mu g/m^3$)	Average Annual Concentration Based on Average Mass Flux[b] ($\mu g/m^3$)	Hazard Quotient (subchronic)	Hazard Quotient (chronic)	Unit Risk Factor	Excess Cancer[c] Risk
TCA	$5.2x10^{-1}$	$4.7x10^{-2}$	$5x10^{-5}$	$4x10^{-5}$	–[d]	–
DCE	--	$3.8X10^{-3}$	--	--	$5.0x10^{-5}$	$2x10^{-7}$
DCA	$3.4x10^{-2}$	$8.0x10^{-4}$	$9x10^{-6}$	$2x10^{-6}$	--	--
TCE	--	$1.7x10^{-3}$	--	--	$1.7x10^{-6}$	$3x10^{-9}$
MC	--	$8.0x10^{-3}$	--	--	$4.7x10^{-7}$	$4x10^{-9}$
PCE	--	$5.2x10^{-6}$	--	--	$5.2x10^{-7}$	$3x10^{-12}$
Maximum Hazard Index[e]	--	--	$5.9x10^{-5}$	$4.2X10^{-5}$	--	--
Maximum Excess Cancer Risk[f]	--	--	--	--	--	$2.1x10^{-7}$

(a) Used for subchronic exposure calculations; values are from the maximum concentration receptor location.

(b) Used for chronic and carcinogenic calculations; values are from the maximum concentration receptor location.

(c) Product of average annual concentration (based on average mass flux) and unit risk factor.

(d) -- denotes information not available and/or not applicable.

(e) Summation of hazard quotients.

(f) Summation of excess cancer risk.

ATTACHMENT C

Revised Appendix E

COLBERT LANDFILL

PHASE I ENGINEERING REPORT

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ACRONYMS AND SYMBOLS

APT	Aquifer Performance Test
Pilot Wells	Pilot extraction wells (CP-S1, CP-W1, CP-E1, CP-E2)
T	transmissivity
K	hydraulic conductivity
S	storativity
S_y	specific yield
c	hydraulic resistance
L	leakage factor
T/S	hydraulic diffusivity
RI	remedial investigation

APPENDIX E

PHASE I AQUIFER PERFORMANCE TEST DATA AND ANALYSIS

This appendix presents data and analyses for the Phase I aquifer performance tests (APTs). Section 1.0 provides a description of the APT procedures. Section 2.0 presents the methods used for analysis and correction of the data, including the governing equations and assumptions. Subsections for each APT are provided in Section 3.0. These subsections include a description of the APT, a discussion of data adjustments (if any) and data analysis, and a presentation of the analysis results. Also included are plots of the water level data for primary observation wells that were analyzed for aquifer parameters (calculations included), and for Pilot Wells and primary observations wells that were not used for analysis. The data plots are included for wells in the vicinity of the pumping wells. Data plots for outlying wells are not included except in the distance-drawdown plots.

1.0 APT PROCEDURES

The Phase I APTs were performed for the four pilot extraction wells (Pilot Wells) CP-S1, CP-W1, CP-E1, and CP-E2, between October 25, 1990 and February 27, 1991. The duration of pumping for each test varied from 7 days for the CP-E2 APT to 13 days for the CP-E1 APT. Each APT included a period of baseline or non-pumping water level data collection, followed by collection of drawdown during a constant-rate pumping test, and concluded with collection of post-pumping aquifer recovery data.

The purpose of the APTs was to obtain data that would provide insight into the behavior of the aquifers tested. Analysis of the response of the aquifers to pumping provides not only a quantification of certain aquifer parameters, but also gives an indication of the rate at which a well can be pumped and the resulting extent of influence and capture zone of the well. Ultimately this information will be used in the design of the Phase II interception/extraction systems and for Phase II treatment system design.

Groundwater levels were recorded using both pressure transducers (coupled to a digital data logger) and hand-held electric water-level indicators. The pressure transducers were installed in the pumping well and nearby monitoring wells and coupled to the digital data loggers. Manual groundwater levels were obtained in the wells with transducers (for calibration purposes), in other wells within or near the expected zone of influence of the pumping well, and in selected wells outside the expected zone of influence. In addition, groundwater levels were recorded in a few wells screened in aquifers other than the one being pumped, to assess the extent of hydraulic connection between the aquifers. All water-level indicators were calibrated to one another; however, the same water-level indicator was generally used for a given well throughout each of the APTs.

In addition to groundwater levels, barometric pressure and precipitation data were collected before, during and after the tests. These data were collected at the onsite meteorological station.

A totalizing flow meter with sweep hand was used to monitor the rate of discharge at the pumping well; the discharge rate was controlled with a globe valve. Groundwater extracted from the pumping well was routed through the Phase I Treatment System (Treatment System) and then discharged to a subsurface infiltration system (South System) or the Little Spokane River (East and West Systems). In order to provide enough water for the operation of the Treatment System, the CP-E2 APT was supplemented with discharge from Pilot Well CP-E1. The rest of the pumping tests consisted of single well discharge. Prior to each APT, the pump was operated long enough to make system adjustments and collect effluent water samples from the Treatment System to verify conformance with the discharge criteria.

2.0 ANALYSIS APPROACH

Following the completion of each test, water level data were transferred to a spreadsheet program for manipulation and analysis. Corrections for partial penetration and dewatering (unconfined aquifers only) were applied to the data (where applicable), and the data plots of time vs. drawdown (and recovery) and time vs. distance were generated for analysis.

Drawdown and recovery data were analyzed to quantify aquifer properties such as transmissivity (T), hydraulic conductivity (K), storativity (S), and specific yield (S_y). Groundwater flow and an aquifer's response to pumping is a function of these parameters. Parameters K and T define how much water will move through the aquifer in unit time under unit hydraulic gradient through a unit area (K), or through a cross section of the aquifer of unit width (T). Hence, T is the product of K times the saturated thickness. Parameters S and S_y define the volume of water released from storage per unit surface area of the aquifer per unit decline in hydraulic head (Kruseman and de Ridder, 1989) for confined aquifers and unconfined aquifers, respectively.

Data from the confined aquifer APT (CP-W1 APT) were analyzed for the hydraulic resistance (c) and leakage factor (L) of the Lacustrine Aquitard. The resistance of an aquitard to vertical flow is given by c, which is a ratio of the aquitard thickness to the vertical hydraulic conductivity of the aquitard. The leakage factor is given by the square root of T times c, and is in units of length. Large values of L indicate a low leakage rate through the aquitard. Aquitard leakage in the Landfill vicinity could be the result of flow from the Upper Sand/Gravel Aquifer, through the Lacustrine Aquitard, to the Lower Sand/Gravel Aquifer; and/or leakage from the thin water-bearing sand layers within the Lacustrine Aquitard to the Lower Sand/Gravel Aquifer.

2.1 Analysis Methods

APT Data were evaluated using a variety of analysis methods, depending on aquifer conditions and data trends. Each of these analyses are based on a number of assumptions and utilize various governing equations to estimate aquifer properties. The following is a brief description of the analyses used for Phase I APTs. Units for variables are specified where conversions are included in the equations, otherwise variables are defined in terms of unitless dimensions of length (L) and time (t).

(1) Theis (1935) drawdown analysis:

The first analysis method to utilize the concept of storage was developed by C.V. Theis in 1935. This method uses log-log plots of drawdown versus time and consists of matching the field data to a type curve. Data values from a common point (termed the match point) are used to provide the necessary input (s, t, $W(\mu)$, μ) to solve the governing equations.

Assumptions:
- Transient flow
- Confined aquifer of infinite areal extent.
- Aquifer is homogeneous, isotropic, and of uniform thickness.
- Piezometric surface is nearly horizontal.
- Discharge is constant.
- Well fully penetrates the aquifer and well storage is negligible.

Equations:

$$T = \frac{Q}{4\pi S}W(\mu)$$
$$\mu = \frac{r^2S}{4Tt}$$

Where:	s =	Drawdown (L) measured in a piezometer at distance r (L) from pumping well.
	Q =	Rate of discharge (L^3/t).
	T =	Transmissivity of aquifer (L^2/t).
	S =	Storativity of aquifer (unitless).
	W(μ) =	Well function.
	t =	Time since pumping began (t).

(2) Cooper and Jacob (1946) drawdown analysis

When values of u are small (r is small and/or t is large), Cooper and Jacob showed that the Theis solution can be approximated by a straight line, semilogarithmic plot of data. Rearranging and adding conversions for different units gives the equations below.

Assumptions: - Same as for Theis (1935) method, and

- u<0.01

Equations:

$$T(ft^2/day) = \frac{35.3Q}{\Delta s}$$
$$S = \frac{Tt_0}{640r^2}$$

Where:	Q =	Rate of discharge (gpm).
	Δs =	Drawdown over one log cycle (ft).
	T =	Transmissivity of aquifer (ft^2/day).
	r =	Distance from pumped well (ft).
	S =	Storativity of aquifer.
	t =	Time since pumping began (days).
	t_0 =	Line intercept at 0 drawdown (mins).
	u =	$[(r^2)(S)]/[4Tt]$

(3) Boulton (1963) drawdown analysis

Drawdown in unconfined aquifers includes a component called delayed yield that results from gravity drainage. This causes unconfined drawdown to deviate from the standard Theis curve. The analysis method developed by Boulton is similar to that of Theis, except that instead of a single type curve there is a family of type curves with different degrees of deviation from the Theis curve. The curve matching procedure is similar to the Theis method.

Assumptions:

- Transient flow.
- Aquifer is unconfined and of infinite areal extent.
- Aquifer is homogeneous and of uniform saturated thickness.
- The water table is horizontal before and during pumping.
- Discharge is constant.
- All wells fully penetrate the aquifer.
- Well storage is negligible.
- Influence of unsaturated zone upon drawdown is negligible.
- $S_y/S > 10$.

Equations:

$$T = \frac{Q}{4\pi s} W(\mu a, \mu b, l)$$

$$S_y = \frac{4Tt(\mu a, \mu b)}{r^2}$$

Where:

s = Drawdown in well (L) at a distance r (L) from the pumped well.

Q = Rate of discharge (L^3/t).

T = Transmissivity of aquifer (L^2/t).

S_y = Specific yield of aquifer (unitless).

t = Time since pumping began (t).

W(μa,l),W(μb,l) = Well function for early and late drawdown, respectively.

(4) Cooper and Jacob (1946) recovery analysis

A semi-log residual drawdown (drawdown remaining after pumping is terminated) versus time since pumping stopped (t') or a ratio of t/t' will result in a straight plot of data, as previously described for drawdown data (Cooper and Jacob 1946).

Assumptions: - Same as for Cooper and Jacob drawdown analysis.

Equation:

$$T(ft^2/d) = \frac{35.3Q}{\Delta s'}$$

Where: T = Transmissivity (ft^2/d) analysis, and

$\Delta s'$ = Residual drawdown over one log cycle (ft).

Q = Rate of recharge = rate of discharge (gpm).

(5) Cooper and Jacob (1946) distance-drawdown analysis

The method developed by Cooper and Jacob for analysis of distance-drawdown data utilizes a semi-log plot of drawdown versus distance. Similar to the Cooper and Jacob drawdown and recovery analyses, a straight line is drawn through the data and the drawdown over one log cycle is determined. The equation for T is essentially equivalent to the steady-state equation developed by Thiem (1906) for steady-state distance-drawdown analysis. An estimate of the pumping well radius of influence can be obtained by projecting a best-fit straight line through the data to the zero drawdown intercept, r_0.

Assumptions: - Same as Theis (1935) method

Equation:

$$T(ft^2/day) = \frac{70.6Q}{\Delta s}$$

$$S = \frac{22.4Tt}{r_0^2}$$

Where: (See Cooper and Jacob drawdown analysis for parameter descriptions and units)

r_0 = Radius of influence = Straight line intercept at 0 drawdown (ft).

(6) Hantush and Jacob (1955) distance-drawdown analysis

Few aquitards are completely impermeable and, as such, some leakage occurs through them. In these cases, the aquifers are not completely confined as assumed for the Theis and Cooper and Jacob methods discussed above. Hantush and Jacob developed a method of analysis for leaky confined aquifers that allows determination of parameters of both the aquifer and the aquitard. The method is an approximation of an equation developed by DeGlee (1930) and uses a semi-log plot of drawdown versus distance. The equation reduces to that used in the Cooper and Jacob distance-drawdown analysis for determination of T. Two additional equations are used to determine leakance (L') and hydraulic resistance (c).

Assumptions:

- The aquifer is leaky.
- The aquifer and aquitard are of infinite areal extent.
- The aquifer is homogeneous, isotropic, and of uniform saturated thickness.
- The piezometric surface is nearly horizontal prior to pumping.
- Discharge is constant.
- The well fully penetrates the aquifer.
- Flow in the aquitard is vertical.
- Drawdown in the unpumped aquifer or aquitard is negligible.
- Flow is steady-state.
- $r/L' < 0.05$, $L' > 3D$ where D = aquifer thickness.

Equations:

$$s = \frac{Q}{2\pi T} Ko(r/L') \text{ DeGlee (1930)}$$

if $r/L < 0.005$ DeGlee's equation can be approximated by:

$$s = \frac{2.30Q}{2\pi T} (\log 1.12\ L'/r)$$

for $r/L'<0.16$ the error is <1 percent. The slope of the straight line is:

$$\Delta s = \frac{2.30Q}{2\pi T}$$

or, adjusting and applying unit conversions:

$$T(ft^2/day) = \frac{70.6Q}{\Delta s}$$

and:

$$c = \frac{(r_0/1.12)^2}{T}$$

$$L' = (Tc)^{1/2}$$

Where:

Q =	Discharge rate (gpm).
Δs =	Drawdown over one log cycle (ft).
T =	Transmissivity of aquifer (ft^2/day).
r_0 =	Line intercept at 0 drawdown (ft).
c =	Hydraulic resistance of the semi-pervious layer (days)
L' =	Leakage factor (ft).

(7) Hantush (1956) drawdown analysis

Another method for analysis of pumping test data for a leaky aquifer was developed by M.S. Hantush in 1956. This method uses a semi-log plot of drawdown versus time. Similar to the Cooper and Jacob drawdown analysis, the data falls on a straight line. However, as steady state is approached and drawdown stabilizes, the data will curve, and the value of maximum drawdown can be approximated.

Assumptions:

- Same as for Hantush and Jacob (1955) except for the last two assumptions therein, and
- Flow is transient.
- The aquitard is incompressible (changes in aquitard storage is negligible).
- It is necessary to extrapolate steady-state drawdown for each piezometer analyzed.

Equations:

$$s_p = 0.5 s_m$$

$$2.30 \frac{s_p}{\Delta s_p} = e^{r/L} K_0(r/L)$$

$$\Delta s_p = \frac{2.30 Q}{4 \pi T} e^{-r/L}$$

$$\frac{r^2 S}{4 T t_p} = \frac{r}{2L}$$

$$c = L^2/T$$

Where:	Q =	Discharge rate (L^3/t).
	s_m =	Maximum drawdown (steady-state) value (L).
	s_p =	Drawdown value used to locate inflection point P (L).
	Δs_p =	Slope of curve at inflection point (L).
	K_0 =	Modified Bessel Function.
	r =	Distance from pumped well (L).
	L' =	Leakage factor (L).
	T =	Transmissivity of aquifer (L^2/t).
	S =	Storativity of aquifer (unitless).
	t_p =	Time of inflection point P (t).
	c =	Hydraulic resistance of semi-pervious layer (t).

(8) Cooper (1963) drawdown analysis

Another method for analyzing leaky aquifers was developed by H.H. Cooper. The method consists of comparing log-log plots of drawdown versus time to sets of type curves, and obtaining match points. The type curves used are for leaky confined aquifers without storage in the semi-pervious layer.

Assumptions:

- Same as for Hantush and Jacob (1955) except for the last two therein, and
- Flow is transient.
- Aquifer is confined.

Equations:

$$T = \frac{Q}{4\pi S} L(\mu,\nu)$$
$$S = 4T \frac{t/r^2}{1/\mu}$$

Where: s = Drawdown in a well (L) located at a distance r (L) from the pumped well.

Q = Discharge rate (L^3/t).

T = Transmissivity of aquifer (L^2/t).

S = Storativity (unitless)

t = Time since pumping began (t) for s to occur.

$L(\mu,\nu)$, $1/\mu$ = x and y axes for type curves.

(9) Hantush (1960) drawdown analysis

This was the first modification of the leaky aquifer theory to account for storage in the semi-pervious layer. The equation can be used for leaky or non-leaky confined aquifers; in the case of the latter application, it is the same as the Theis solution. Log-log plots of the field data are matched to type curves developed by McClellan (1961) and presented in Lohman (1979). The solution of S is identical to that for the Theis solution.

Assumptions:
- same as for Cooper (1963), and
- with or without storage in the semi-pervious layer.

Equations:

$$T = \frac{Q}{4\pi s} H(\mu,B)$$
$$S = \frac{4Ttu}{r^2} \text{ or } \frac{4Tu}{r^2/t}$$

Where: T, W, S, t,μ, and r are the same as for Cooper (1963)

$H(\mu,B)$ = Well function

(10) Jenkins and Prentice (1982) drawdown analysis

Groundwater flow in fractured aquifers includes, or consists entirely of, linear fracture flow. All the previously discussed analysis methods are of radial flow conditions in porous media and are not applicable to linear flow systems. Jenkins and Prentice developed a method to determine the location of a fracture and to determine the hydraulic diffusivity (T/S) of the aquifer. The method utilizes the fact that in linear flow systems, plots of drawdown versus $t^{1/2}$ will define a straight line.

Assumptions:

- Flow is transient.
- Aquifer is confined.
- Fracture is homogeneous, isotropic and of infinite areal extent.
- Aquifer is fully penetrated by a single vertical fracture.
- Storage in fracture can be neglected, and its horizontal extent is finite.
- Resistance to flow within the fracture is negligible.
- Flow within the fracture is laminar and linear toward the pumped well.
- Water from the aquifer enters the fracture at the same rate per unit area.

Equations:

$$(t_{0B}/t_{0A})^{1/2} = X_B/X_A$$

$$\Theta_B = \tan^{-1}\frac{r_A(t_{0B})^{1/2}\sin\Delta\Theta}{r_B(t_{0A})^{1/2}-r_A(t_{0B})^{1/2}\cos\Delta\Theta}$$

$$X_A = r_A\sin\Theta_A$$

$$X_B = r_B\sin\Theta_B$$

$$\frac{T}{S} = \frac{\pi X^2}{4t_0}$$

Where:

	X =	Distance of well from extended well (fracture) (L).
	t_0 =	Line intercept at 0 drawdown (t).
	Θ =	Angle between fracture and line connecting well and pumped well (deg).
	$\Delta\Theta$ =	Difference between Θ_A and Θ_B (deg).
	T/S =	Hydraulic diffusivity (L^2/t).
	A and B =	Observation well designations.

(11) Gringarten and Witherspoon (1972) drawdown analysis

In a system containing a vertical fracture, Gringarten and Witherspoon recognized that T will be greater parallel to the fracture than perpendicular to it. They developed three families of type curves that are selected based on the observation well position with respect to the fracture. Log-log plots of drawdown versus time are compared to the type curves and match points are noted [see Kruseman and de Ridder (1989) for type curves].

Assumptions:
- same as for Jenkins and Prentice, and
- Aquifer is of uniform thickness.
- Well fully penetrates the aquifer.
- Discharge is constant.
- The piezometric surface is horizontal prior to pumping.

Equations:

$$T = \frac{Q}{4\pi s} F(U_{vf}, r')$$

$$S = \frac{Tt}{U_{vf} X_f^2}$$

$$X_f = r/r'$$

$$U_{vf} = (U_{vf}/r')(r')$$

$$r' = (x^2 + y^2)^{1/2}/X_f$$

Where:

T = Transmissivity of aquifer (L^2/t).

Q = Rate of discharge (L^3/t).

s = Drawdown (L) in a well at distance r (L) from the pumping well.

t = Time since pumping started (t).

r′ = Type curve used.

$F(U_{vf}, r')$, U_{vf}/r' = y and x axes of type curves.

X_f = Half length of vertical fracture (L).

x,y = Distance between observation well and pumping well along axes (L).

Note: Where r′>5 radial flow occurs and the equation becomes identical to that of Theis.

(12) Jacob (1944) correction for dewatering

Gravity drainage of interstices decreases the saturated thickness and therefore the coefficient of transmissivity of the aquifer under water table conditions. Observed values of drawdown in unconfined aquifers should be adjusted for the decrease in saturated thickness before the data can be used in analysis methods presented above. The correction is generally applied to data where the drawdown exceeds 10 percent of the saturated thickness of the aquifer.

Equation:

$$s' = s - (s^2/2B)$$

Where:

s = Observed drawdown (ft).

s' = Corrected drawdown (ft).

B = Saturated thickness of the aquifer.

(13) Butler (1957) correction for partial penetration

Groundwater flow lines are distorted around a pumping well that only partially penetrates the aquifer. Data from pumping wells and observation wells within a critical distance that partially penetrate an aquifer must be corrected before being used in the methods discussed above. The correction is generally applied to wells within a distance equivalent to two times the aquifer thickness of the pumping well, beyond this point the effects of partial penetration are considered to be negligible.

Equation:

$$s' = C_p s$$

Where:

s = Observed drawdown (L).

s'= Corrected drawdown (L).

C_p = Partial penetration constant obtained from tables which use the following parameters:

- r [distance from pumped well to observation well (L)]
- a fractional penetration
- r_e virtual radius (L) of cone of depression, assumed to be 10,000 ft for confined and 1,000 ft for water table conditions.

(14) Driscoll (1986) estimation of well efficiency

Driscoll provides a method of estimating well efficiency from pumping test data based on well specific capacity (specific capacity is the ratio of discharge to drawdown). Well efficiency is calculated using the following equation:

$$e = \frac{(\frac{Q}{s})\ meas}{(\frac{Q}{s})\ theor}\ (100)$$

Where: e = well efficiency (percent)

(Q/s)meas = measured specific capacity, with s obtained at t=1 day

(Q/s)theor = theoretical specific capacity based on the equation:

(Q/s)theor = T/2,000

Where Q = pumping rate (gpm)

s = drawdown (ft)

T = transmissivity (gpd/ft)

Driscoll recommends using the average T calculated from early time data for the pumping well and transmissivity estimated from observation well(s). For the purposes of this report, the T value used for analysis is the average of T estimated for the pumping well and the average T provided in the appropriate summary table (Tables E-1 through E-3).

3.0 PHASE I APT ANALYSES

Phase I APT operational information, data plots, data analyses, and aquifer design parameters are presented in this section for the CP-S1, CP-W1, CP-E1, and CP-E2 APTs. The analysis methods utilized are those discussed in the preceding section, and that are applicable to the hydrogeologic conditions for each APT location.

3.1 CP-S1 APT Analysis Summary

Data collected during the CP-S1 APT for the Upper Sand/Gravel Aquifer consists of drawdown data collected during a constant-rate pumping test and recovery data recorded after pumping was terminated. The drawdown data were collected from the time the pump was turned on at 9:35 on October 25, 1990, until the pump was turned off at 14:02 on November 5, 1990. Recovery data were recorded from 14:02 on November 5, 1990, until 7:15 on November 12, 1990. The average pumping rate at CP-S1 was 95 gpm. Transducers were used in CP-S1, CD-33, and CD-30 Upper Sand/Gravel Aquifer Wells to record drawdown and recovery data. Manual water level readings were collected at Friedrichsen, CD-31, and CD-32 Upper Sand/Gravel Aquifer Wells. Static water levels were monitored prior to the start of the test from 11:34 on October 23, 1990, to 9:35 on October 25, 1990. Barometric pressure and precipitation data were recorded prior to and throughout the test.

Data from the transducers and manual recordings were imported into a spreadsheet program. The time and probe readings were adjusted relative to the static (t_0) readings to get t (time since pumping started), and s (drawdown). Time since pumping stopped (t′) was based upon the time the pump was shut off. The drawdown at CP-S1 was greater than 10 percent of the aquifer saturated thickness so a dewatering correction was applied (see Equation No. 12 from Section 1.2).

There does not appear to be a relationship between the barometric pressure readings and the baseline water levels. During the pre-test period water levels rose 0.1 feet and the barometric pressure dropped roughly 10 millibars (mb), but these fluctuations did not correlate to each other. During the pumping period the barometric pressure fluctuated, varying a maximum of 10 mb. The drawdown plots did not show any significant fluctuations corresponding with the swings in barometric pressure and, consequently, the data were not corrected for barometric pressure.

Data plots consist of semi-log and log-log drawdown plots (s vs. t), semi-log recovery plots (residual s vs t/t′), and semi-log distance-drawdown plots (s vs r).

Analyses for the CP-S1 APT consist of Cooper and Jacob (1946) straight line analysis for the semi-log drawdown, recovery, and distance-drawdown plots (see Equation Nos. 2, 4, and 5 from Section 2.1), and Boulton (1963) curve-matching for the log-log drawdown plots (see Equation No. 3 from Section 2.1). All analyses were performed for Wells CD-30, CD-33, and Friedrichsen. Analyses were not performed at CP-S1 due to pumping well boundary conditions (such as well loss). Data plots for these wells, including match points and calculations (where applicable) are shown in Figures E-1 through E-12.

The Cooper-Jacob equation is an approximation of the Theis equation and uses the fact that as μ becomes small (r is small and t is large) parts of the Theis equation become negligible and can be ignored. Therefore, in applying the Cooper-Jacob method a test must be made for μ; μ must be <0.01 (Kruseman and deRidder 1989), or <0.05 (Lohman 1979). The u-test indicated the method to be marginally valid for the observation wells analyzed. The recovery analysis was applied to late recovery data, where t/t' is small, and the effects of delayed yield have dissipated and residual drawdown falls on a straight line (Kruseman and deRidder 1989). The Boulton method uses the Theis equation with the exception that the well function contains a term for delayed yield, a characteristic of unconfined aquifers, and therefore the type curves deviate from the Theis curve. The log-log plots for the Boulton analysis were developed at the same scale as the type curves presented in Lohman (1979).

The distance-drawdown analysis assumes transient conditions but uses an equation for T which is essentially the same as the basic equation developed by Thiem (1906) for steady-state conditions. The distance-drawdown plot utilizes data from the end of the constant-rate pumping test, and was used to determine T and Sy. The distance drawdown plot was also used to estimate the radius of influence of CP-S1 by extending the line to the 0-ft drawdown point and reading the value for r_0 at that point.

The method described is Section 2.1 (14) was utilized to estimate a well efficiency of 64 percent. Well efficiency calculations are shown on Figure E-11.

A saturated thickness of 19 ft was used to estimate K; this was based on the geologic profile and saturated thickness for CP-S1 provided in Appendix B. A summary of the results for the Cooper, Jacob, and Boulton analyses are included in Table E-1. The estimated values for T range from 9600 to 18100 ft^2/day, with an arithmetic mean (average) of 12000 ft^2/day. The values for K range from 610 to 1100 ft/day, with an arithmetic mean (average) of 640 ft/day. These values fall within the range of K for coarse sand to medium gravel. Only two of the eleven T estimates are above the average, and, therefore, the average T is considered a reasonable upper bound. Inspection of the data indicate 10000 ft^2/day and 530 ft/day represent reasonable lower bound for T and K, respectively.

The Sy values vary from 0.1 to 0.25, with the average 0.20. This average value is within the range expected for coarse sand and gravel. The storage coefficients from the early time Boulton analyses were not included in this average because they are more closely related to S than Sy, and do not adequately reflect storage from delayed yield. Specific yield could not be determined from the recovery plots. The distance-drawdown plot (shown on Figure E-10) indicates a radius of influence (r_0) of 1000 feet from CP-S1 during pumping, which appears reasonable for an unconfined aquifer with the transmissive properties of the Lower Sand/Gravel Aquifer.

3.2 CP-W1 APT Analysis Summary

Data collected during the CP-W1 APT for the Lower Sand/Gravel Aquifer consists of drawdown data collected during a constant-rate pumping test, and limited recovery data recorded after pumping terminated. The drawdown data were collected from the time the pump was turned on at 09:05 on November 13, 1990, until pumping terminated on November 23, 1990, at approximately 11:00 (based on meter readings) due to a power failure. The average pumping rate was 220 gallons per minute. Greater than 90 percent of aquifer recovery occurred in the first two hours after pumping stopped (when it was not known that the pump was off); recovery data were not collected during this period. However, recovery data were collected from about 13:00 on November 23 until November 26, 1991.

Pressure transducers were used in Lower Sand/Gravel Aquifer Wells CP-W1, CD-46C2, CD-47C2, and CD-2L. Manual water level readings were collected at Lower Sand/Gravel Aquifer Wells CD-3L, CD-5, CD-6L, CD-21, CD-23D1, CD-26, CD-44, CD-45, and the C1, C2, and C3 wells at CD-40, CD-41, CD-42, and CD-43. Manual readings were also taken at CS-4 in the Upper Aquifer, and at CD-4L and CD-20E2 in the Basalt Aquifer. Static water levels were monitored prior to the start of the test from 14:54 on November 12, 1990, until 9:04 on November 13, 1990. Barometric pressure and precipitation data were recorded prior to and throughout the test.

Data from the transducers and manual recordings were processed as for CP-S1. Partial penetration corrections (see Equation No. 13 in Section 2.1) were applied to the data from CP-W1 and CD-47. Partial penetration corrections were not necessary for the other wells since their distance from the pumping well is greater than two times the aquifer thickness. There were no significant fluctuations in water levels during the test that would correlate with barometric changes; therefore, corrections for barometric pressure were not necessary.

Drawdown reached apparent steady-state conditions early in the test, indicating recharge to the aquifer. This recharge probably is the result of the Lacustrine Aquitard leakage, and analyses were applied to quantify the leakage that occurred through the overlying aquitard.

No drawdown was observed in the Upper Sand/Gravel Aquifer Well (CS-4) during the CP-W1 APT. Drawdown was observed (about 0.3 ft) in the Basalt Aquifer in CD-4L, but not in CD-20E2, suggesting these basalt units may not be hydraulically connected.

The drawdown data were plotted as for CP-S1 and are presented on Figure E-13 through E-24. Analyses methods included Cooper and Jacob (1946) straight line method for semi-log plots of drawdown and distance-drawdown (see Equation Nos. 2 and 5 in Section 2.1), Theis (1935) for log-log plots of drawdown (see Equation No. 1 in Section 2.1), and methods for analysis of leaky aquifer conditions developed by Hantush and Jacob (1955), Hantush (1956), Cooper (1963), and Hantush (1960) (see Equation Nos. 6, 7, 8, and 9 in Section 2.1). The semi-log and log-log analysis method that do not account for leaky aquifer conditions were applied to early time data, except for distance-drawdown analyses, which utilized late time (t=8.0 days) data. Due to the unanticipated termination of pumping for the CP-W1 APT, and the resulting lack of early recovery data, recovery analysis was not used. All other analyses were applied to CD-2, CD-46, and CD-47. Only the Cooper and Jacob (1946) drawdown method was used for CD-41C2 because of limited data. Analyses were not performed for CP-W1 data due to boundary effects (such as well loss) that impact pumping well data. Individual analyses were also not performed for the remaining wells due to the limited number of data points and minimum drawdown, although some of the locations were used in the distance-drawdown analysis.

A summary of the results are included in Table E-2. The estimates for T range from 26000 to 71000 ft^2/day with an average value of 40000 ft^2/day. In general, the higher T estimates are from the analysis methods that do not account for leakage. Based on a saturated thickness of 175 ft obtained from the geologic profile and depth to water for CD-47 (presented in Appendix B), K ranged from 140 to 350 ft/day with an average of 230 ft/day, which is in the range of a medium to coarse sand. S ranged from 0.0001 to 0.034, with an average of 0.0004; the value of 0.034 from the distance-drawdown analysis may not be representative, and is not included in this average. The distance-drawdown plot (as shown on Figure E-23) indicates a radius of influence of 9500 feet for CP-W1, which appears reasonable for a confined aquifer with the transmissive properties of the Lower Sand/Gravel Aquifer. CP-W1 well efficiency is estimated at 82 percent using the method described in Section 2.1 (14). Well efficiency calculations are shown on Figure E-24. Only five of the eighteen analysis methods estimated a T value higher than the average, and, therefore, the average values of 40,000 ft^2/day and 230 ft/day are considered reasonable upper bound estimates of T and K, respectively. Inspection of the data indicate 30,000 ft^2/day and 170 ft/day represent reasonable lower bounds for T and K, respectively.

3.3 CP-E1 APT Analysis Summary

Data collected during the CP-E1 APT for the Lower Sand/Gravel Aquifer consist of drawdown data collected during a constant-rate pumping test and recovery data recorded after pumping was terminated. The drawdown data were collected from the time the pump was turned on at 11:35 on January 22, 1991, until the pump was turned of at 12:05 on February 4, 1991. The average pumping rate was 202 gpm. Recovery data were recorded from the time the pump was shut off until February 8, 1991, when 99 percent recovery had been achieved in CP-E1. Transducers were used in Lower Sand/Gravel Aquifer Wells CP-E1, CD-24C2, CD-21C1, CD-21C3 and the Wilson private well. Manual readings were recorded at Lower Aquifer Sand/Gravel Aquifer Wells CD-1, CD-41C2, CD-42C2, CD-42C3, CD-43C2, CD-44C2, CD-45C2, CD-46C2, CD-47C2, and the Wahl private well. Manual readings were also collected for Basalt Aquifer Wells CD-4L, CD-7L, and CD-8L, and in the Upper Sand/Gravel Aquifer Well CS-4. Barometric pressure and precipitation data were recorded prior to and throughout the test.

Data from the transducers were processed as previously described for CP-W1 and CP-S1 APTs. Partial penetration corrections were applied to Wells CP-E1, CD-24, CD-21C3, and Wilson (see Equation No. 13, Section 2.1). The critical distance for partial penetration is about 320 feet and all drawdown data from wells beyond this distance were not corrected. In contrast to CP-W1, the Lower Sand/Gravel Aquifer in the area of CP-E1 is predominantly unconfined because the base elevation of the Lacustrine Aquitard increases from west to east. Drawdown in the wells during the CP-E1 test, however, did not exceed 10 percent of the saturated thickness; thus, no dewatering correction was applied.

Some wells recovered to levels above the initial static level, suggesting barometric influence or recharge. There was no direct correlation between barometric fluctuations or precipitation during the test and recorded water levels; therefore, no correction was applied.

Drawdown and recovery data were plotted as for CP-S1 and are presented on Figures E-25 through E-42. Log-log plots of drawdown data were analyzed using the Boulton method (see Equation No. 3 in Section 2.1), and semi-log plots of the drawdown, recovery, and distance-drawdown data were analyzed by the Cooper and Jacob methods (see Equations Nos. 2, 4, and 5 in Section 2.1).

The data plots show discharge boundary condition influences from delayed yield and from a discharge boundary (or boundaries); delayed yield influences are most apparent in Wells CD-21C3 and CD-24C2, and discharge boundary conditions are observable in Wells CD-21C1 and Wilson. The combination of limited drawdown (generally a foot or less at observation wells) and complex boundary conditions make data interpretation ambiguous for the CD-21C1 and Wilson wells. Consequently, aquifer parameters were not estimated using data from these wells, except in the distance-drawdown analysis.

The log-log curve matching analyses appear to provide the best estimate of aquifer parameters and were used for the data from CD-1, CD-21C3, and CD-24. Semi-log analyses were also performed for these wells, but is limited to early time data for drawdown and late time data for recovery. Because of the combined (and opposite) impacts of delayed yield and a discharge boundary condition, the use of other portions of the data plots for semi-log analysis is considered inappropriate. Analyses of CP-E1 data were not performed due to boundary effects (such as well loss) that impact pumping well water level data.

Methods developed by Ferris et al (1962) and Stallman (1963) were used in an attempt to analyze the discharge boundary location(s). However, delayed yield influences interfere with the discharge boundary effects, and the boundary location(s) could not be clearly identified.

There appears to be a hydraulic connection between the lower aquifer and the Basalt Aquifer as evidence by drawdown at Basalt Aquifer Wells CD-4L and CD-8L (see Figure E-41), but no drawdown was observed in the Basalt Aquifer at CD-7L. Also, no drawdown was observed in the Upper Sand/Gravel Aquifer at CS-4, which is consistent with expectations, based on present understanding of the groundwater flow system.

A summary of the analysis results are included in Table E-3. The estimates of T range from 9700 to 22000 ft^2/day, with an average of 14000. Based on a saturated thickness of 100 ft obtained from the geologic profile and depth to water for CP-E1, values for K range from 100 to 220 ft/day, with an average of 140 ft/day; this is in the range of a medium to coarse sand. The average Sy is 0.16, which is within the typical range of values for an unconfined aquifer. Four of the ten analyses estimate T values greater than the average. However, all of these higher estimates are based on straight line analysis methods that do not allow evaluation of certain aquifer conditions, such as delayed yield, and may tend to overestimate T. Consequently, the average T and K values are considered reasonable upper bound estimates. Inspection of the data indicate 10,000 ft^2/day and 100 ft/day represent reasonable lower bounds for T and K, respectively.

The distance-drawdown plot indicates a radius of influence of 3500 ft for Well CP-E1, based on a pumping period of 7.3 days. This estimate of radius of influence appears reasonable for a semiconfined aquifer, or an aquifer that transitions between confined and unconfined conditions (as does the Lower Sand/Gravel Aquifer in CP-E1 vicinity). Using the method described in Section 2.1 (14), well efficiency is estimated to be 78 percent. Well efficiency calculations are shown on Figure E-40.

3.4 CP-E2 APT Analysis Summary

An initial pumping test was begun at CP-E2 on February 14, 1991, at 10:05, at a discharge rate of 15 gpm. However, the pump was turned off on February 15, 1991, at 11:05, when available drawdown was exceeded.

After letting the groundwater recover to near static levels, the CP-E2 APT was re-initiated on February 20, 1991, at 10:35, at a reduced flow rate of 4.5 gpm. Well CP-E1 was also pumped during the CP-E2 APT (at 150 gpm) to supply enough water for the treatment system operation. Drawdown data were recorded until the pumps were turned off on February 27, 1991, at 11:05. Recovery data were recorded from the time the pumps were turned off until March 13, 1991, at which point the groundwater in Pilot Well CP-E2 had recovered to about 90 percent of pre-pumping levels.

Pressure transducers were used in Basalt Aquifer Wells CP-E2, CD-25E1, CD-20E1, and CD-20E2. Manual readings were recorded for Basalt Aquifer Wells CD-4L, CD-7L, CD-8L, CS-14L, and the (b) (6) [redacted] private wells. Manual readings were also measured for CD-6U in the Upper Sand/Gravel Aquifer; for CD-6L and CD-26C1 in the Lower Sand/Gravel Aquifer; for CD-22D1 and CS-14U in the Latah Aquitard; and for CD-23B1 in the Lacustrine Aquitard. Barometric and precipitation data were recorded prior to and during the test.

Data were processed as for the CP-S1, CP-W1, and CP-E1 APTs, and data plots for the CP-E2 APT are provided in Figures E-43 through E-58. No corrections were applied to the data. Drawdown could only be identified at Wells CP-E2, CD-25, CD-20E1, CD-20E2, CD-4L, and

CD-7L. The drawdown at CD-4L appears to be primarily (or wholly) attributable to the pumping from Pilot Well CP-E1 during the CP-E2 APT, based on the response of CD-4L during the CP-E1 APT. Thus, no direct hydraulic connection is apparent between the Basalt Aquifer at the CP-E2 location, and the Upper and Lower Sand/Gravel Aquifers and the Latah Aquitard, based on CP-E2 APT results. Basalt Aquifer drawdown data from the CP-W1, CP-E1, and CP-E2 APTs indicate a hydraulic separation between different portions of the Basalt Aquifer, which supports the presence of independent Basalt landslide blocks or possibly separate Basalt flow layers.

Semi-log plots of drawdown vs. time showed distinct and continuous curvature instead of straight lines, suggesting that flow was linear rather than radial (Lewis and Burgy 1964) and/or (possibly) multiple discharge boundary conditions; linear flow is typically associated with fractured rock, such as the Basalt Aquifer. Furthermore, arithmetic plots of drawdown vs. the square root of time produce straight lines, a phenomenon discussed by Jenkins and Prentice (1982) as being indicative of linear rather than radial flow. Methods of analyses used on the previous pumping tests were designed for radial flow systems. The hydrogeologic conditions at CP-E2 appear to differ significantly from the assumptions for radial flow methods, and these methods were not applied to the CP-E2 APT data.

The majority of the analysis methods identified for linear low systems were developed for the petroleum industry, and have only recently been applied to groundwater. A review of analysis methods for linear flow systems indicates solutions, typically, either assume that a single vertical fracture exists or that the aquifer is so extensively fractured that flow can be described by a dual porosity model. The Basalt Aquifer is likely to be somewhere between these extremes, and is limited in size, which deviates from the assumption of infinite areal extent inherent in all of the methods reviewed. The data were analyzed using several of the available methods for linear flow to evaluate whether aquifer response was similar enough to that predicted to allow assessment of aquifer parameters; the Jenkins and Prentice method and Gringarten and Witherspoon method appear to adequately approximate observed aquifer response. Methods for analyzing double porosity systems, including those developed by Bourdet and Gringarten (1980), Kazemi et al (1969), and Warren and Root (1963), were evaluated but were determined to be inapplicable based on CP-E2 APT data plots.

Jenkins and Prentice (see Equation No. 10 in Section 2.1), developed an analytical approach based on the conceptual model that a pumping well penetrating a fracture will behave as an extended well; that is, the entire fracture will act as a pumping well and flow will be linear towards the fracture rather than radial toward the well. Under these conditions, drawdown in the observation wells is directly related to their distance from the fracture. Their method, which uses the semi-log plot of drawdown versus $t^{1/2}$, was applied to one pair of wells (CD-20E1 and CD-20E2) to locate the fracture and determine hydraulic diffusivity (T/S) of the aquifer; the results were used to analyze a second pair of wells (CD-25E1 and CD-20E2). Analysis of data from Wells CD-20E1 and CD-20E2 provided diffusivity values in the range of 5200 ft^2/day while analysis of Well CD-25E1 data gave a much higher value of 162500 ft^2/day. No unique fracture location fit the data, suggesting (1) one of the wells may not be on the same side of the fracture, (2) the fracture is not a single straight, planar, or vertical feature, or (3) flow to the fracture is not perpendicular due to anisotropic properties of the aquifer (Smith and Vaughan 1985).

A method developed by Gringarten and Witherspoon (see Equation No. 11 in Section 2.1) is similar to that of Jenkins and Prentice, except that it assumes a maximum transmissivity parallel to the fracture(s) and a minimum transmissivity perpendicular to it. This anisotropy would be typical of a system of parallel fractures. Log-log plots of drawdown versus time are matched to type curves for various geometries. Since the fracture location is unknown, a trial and error method is applied to obtain solutions consistent with the data. Type curves were matched to log-log plots of drawdown versus time for Wells CP-E2, CD-25E1, CD-20E1 and CD-20E2. The plots for CD-25 and CD-20E1 were found to match type curves for two different geometries, while the other two plots matched only one. The curves matched well except for late portions of the plots where drawdown continued linearly for the data and the type curves took on radial flow trends. This deviation can be explained by the possible presence of minor fractures that affect the drawdown response of the wells (Mousli et al. 1982), or discharge boundary conditions.

A summary of analysis results are provided in Table E-4. Using match points from the data plots and type curves, T is estimated to range from 11 to 33 ft^2/day with an average of 25 ft^2/day. Bulk K is estimated to range from 0.3 to 0.8 ft/d, although hydraulic conductivity along individual fractures may be significantly higher. Storativity ranged from 0.0001 to 0.045 averaging 0.01. The T and K values are significantly lower than that estimated in the remedial investigation for the (b) (6) well.

The transmissivity estimate in the remedial investigation for the Basalt Aquifer is based on recovery analysis of a short-term pumping test of the (b) (6) private well. The remedial investigation analysis estimated T using the late time recovery data, although use of the intermediate recovery data appears to be more appropriate. Re-evaluation of the remedial investigation data using the intermediate recovery values (see Figure E-58) results in an estimate of 26 ft^2/d for T, similar to that estimated based on the EP-E2 APT data. Although the recovery analysis method used (Cooper and Jacob 1946) may not be appropriate for the Basalt Aquifer (due to fracture flow), the re-evaluation estimate for T appears to provide a better estimate of T than that provided in the remedial investigation.

Although the nature of the Basalt Aquifer is not fully understood, analysis of the pumping test data suggest that flow in the aquifer may be linear rather than radial and that multiple discharge boundary conditions may be present. Regardless of the governing flow conditions, the estimated T is low for an "aquifer", and long-term groundwater extraction rates are anticipated to be minimal for the Basalt Aquifer.

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TABLE E-1

COLBERT LANDFILL RD/RA
CP-S1 APT AQUIFER PARAMETER SUMMARY

Observation Well	Test Type	T (ft^2/d)	K[(a)] (ft/d)	Sy[(b)]	Analysis Method[(c)]	Comments
CD-30	Drawdown	12000	630	0.25	(2)	u=0.046
	Drawdown	14000	740	0.20	(3)	Late time
	Recovery	11000	580	---	(4)	
CD-33	Drawdown	11000	580	0.20	(2)	u=0.004
	Drawdown	18000	950	---	(3)	Early time
	Drawdown	12000	630	0.16	(3)	Late time
	Recovery	11000	580	---	(4)	
(b) (6)	Drawdown	12000	630	0.10	(2)	u=0.04
	Drawdown	10000	530	---	(3)	Early time
	Recovery	9600	510	---	(4)	
Distance-Drawdown	Drawdown	11000	---	0.27	(5)	t=11.12 days
Average		12000	640	0.20		

(a) Based on 19 ft saturated thickness.

(b) Specific yield from late time data only.

(c) Analysis Method number from Section 2.1.

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TABLE E-2

COLBERT LANDFILL RD/RA
CP-W1 APT AQUIFER PARAMETER SUMMARY

Observation Well	Test Type	T (ft^2/d)	K[a] (ft/d)	S	Analysis Method[b]	Comments
CD-2	Drawdown	38000	210	0.0005	(1)	
	Drawdown	31000	180	0.0004	(2)	u=0.05
	Drawdown	27000	150	0.0004	(7)	L'[c]=4200 ft, c[d]=650 days
	Drawdown	26000	150	0.0005	(8)	
	Drawdown	24000	140	0.0006	(9)	
CD-46	Drawdown	61000	350	0.0004	(1)	
	Drawdown	71000	400	0.0001	(2)	u=0.0007
	Drawdown	69000	390	0.0002	(7)	L'=34000 ft, c=16800 days
	Drawdown	37000	210	0.0004	(8)	
	Drawdown	37000	210	0.0004	(9)	
CD-47	Drawdown	37000	210	0.0004	(1)	
	Drawdown	46000	260	0.0002	(2)	u=0.0008
	Drawdown	45000	260	0.0002	(7)	L'=9700 ft, c=2100 days
	Drawdown	31000	180	0.0003	(8)	
	Drawdown	34000	190	0.0003	(9)	
CD-41C2	Drawdown	35000	200	0.0007	(1)	u=0.01
Distance-Drawdown	Drawdown	39000	---	0.034	(5)	t=8.0 days
	Drawdown	(e)	---	---	(6)	L'=2100 ft, c=550 days
Average		40000	230	0.0004[f]		

(a) Based on 175 ft saturated thickness.

(b) Analysis Method number from Section 2.1.

(c) L = Leakage factor.

(d) c = Hydraulic resistance of semi-pervious layer.

(e) Same equation and plot used in Cooper-Jacob distance-drawdown calculation.

(f) Does not include distance drawdown estimate in average.

TABLE E-3

COLBERT LANDFILL RD/RA
CP-E1 APT AQUIFER PARAMETER SUMMARY

Observation Well	Test Type	T (ft^2/d)	K[a] (ft/d)	Sy	Analysis Method[b]	Comments
CD-24	Drawdown	11000	110	---	(3)	Early data
	Drawdown	13000	130	0.18	(3)	Late data
	Drawdown	22000	270	---	(2)	
	Recovery	9900	100	---	(4)	
CD-21C3	Drawdown	13000	130	---	(3)	Early data
	Drawdown	9700	100	0.14	(3)	Late data
	Recovery	15000	150	---	(4)	
CD-1	Drawdown	14000	140	---	(3)	Early data
	Drawdown	16000	160	---	(2)	
Distance-Drawdown	Drawdown	18000	---	---	(5)	t=7.4 days
Average		14000	140	0.16		

(a) Based on 100 ft saturated thickness.

(b) Analysis Method number from Section 2.1.

TABLE E-4

COLBERT LANDFILL RD/RA
CP-E2 APT AQUIFER PARAMETER SUMMARY

Observation Well	Test Type	T (ft^2/d)	K[a] (ft/d)	S	T/S[b]	Xf[c]	X[d]	Type Curve	Analysis Method[e]
CP-E2	Drawdown	27	0.7	---	---	---	---	r'=1.02, y=0	(11)
CD-20D1	Drawdown	27	0.7	0.03	880	13	---	r'=1.50, y=0	(11)
	Drawdown	11	0.3	0.002	5300	286	---	r'=0.07, x=0	(11)
	Drawdown	26	0.6	0.005	5300	___	19		(10)
CD-20D2	Drawdown	23	0.6	0.045	500	22	---	r'=1.02, y=0	(11)
	Drawdown	26	0.6	0.005	5300	---	5		(10)
CD-25	Drawdown	33	0.8	0.004	8200	58	---	r'=1.1, y=0	(11)
	Drawdown	33	0.8	0.0001	65600	1200	---	r'=0.05, x=0	(11)
	Drawdown	33	0.8	0.0002	163000	---	62		(10)
Average		27	0.7	0.01					

(a) Based on 40 ft saturated thickness.

(b) T/S = Hydraulic diffusivity (ft^2/day).

(c) Xf = Fracture half length (ft).

(d) X = Distance to fracture (ft).

(e) Analysis Method number from Section 2.1.

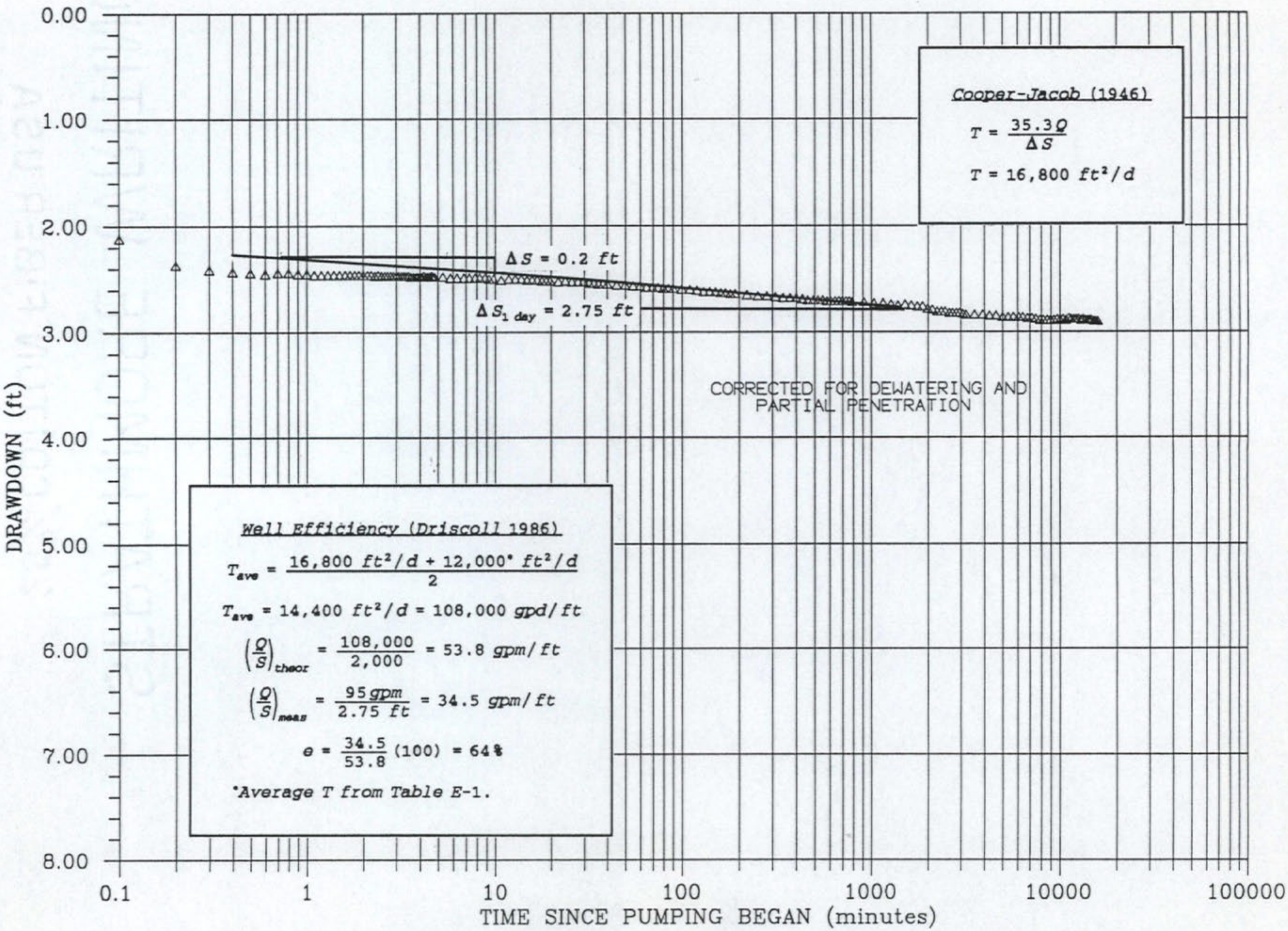

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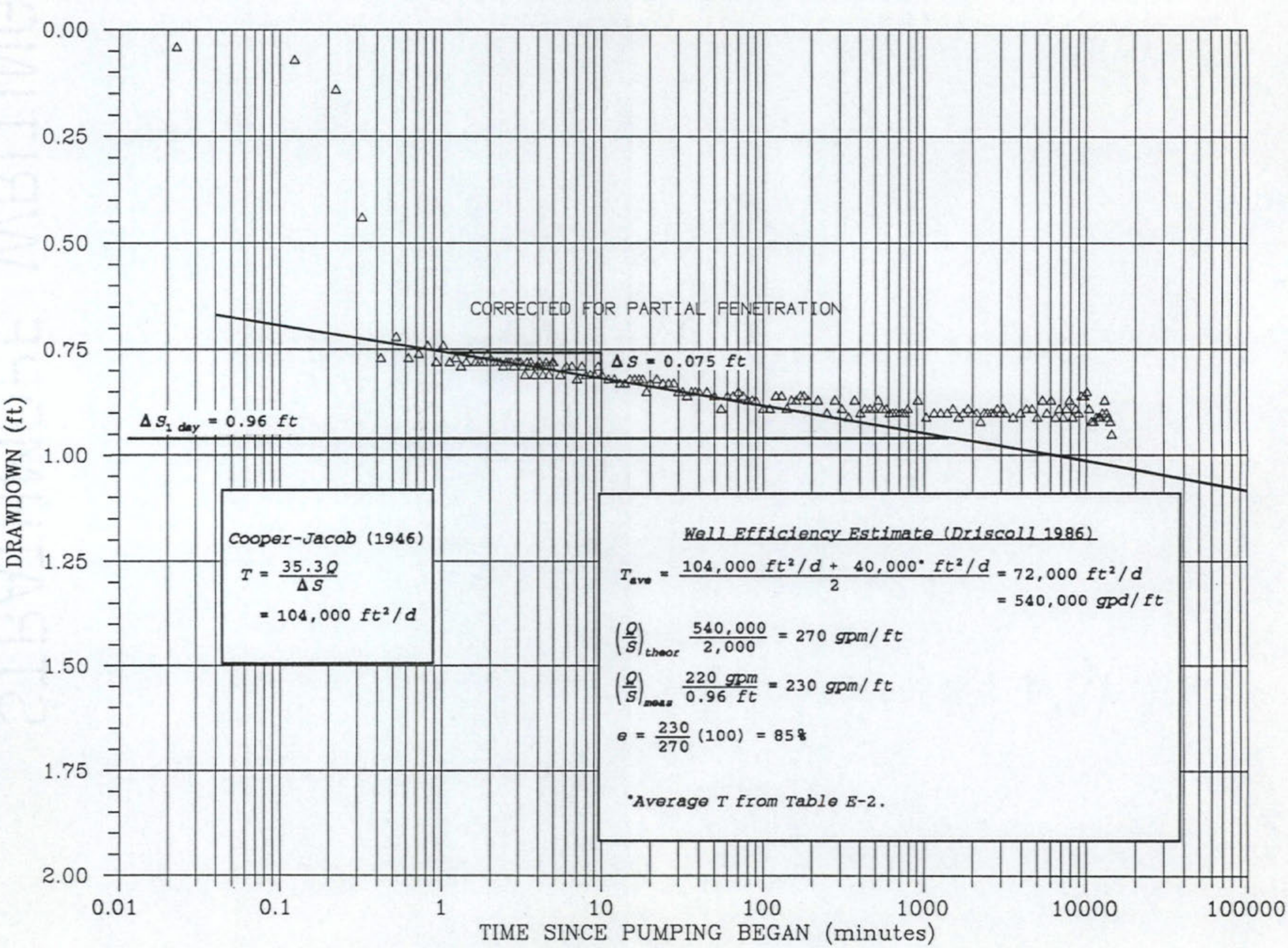
Figure E-11



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CP-W1 Drawdown Data

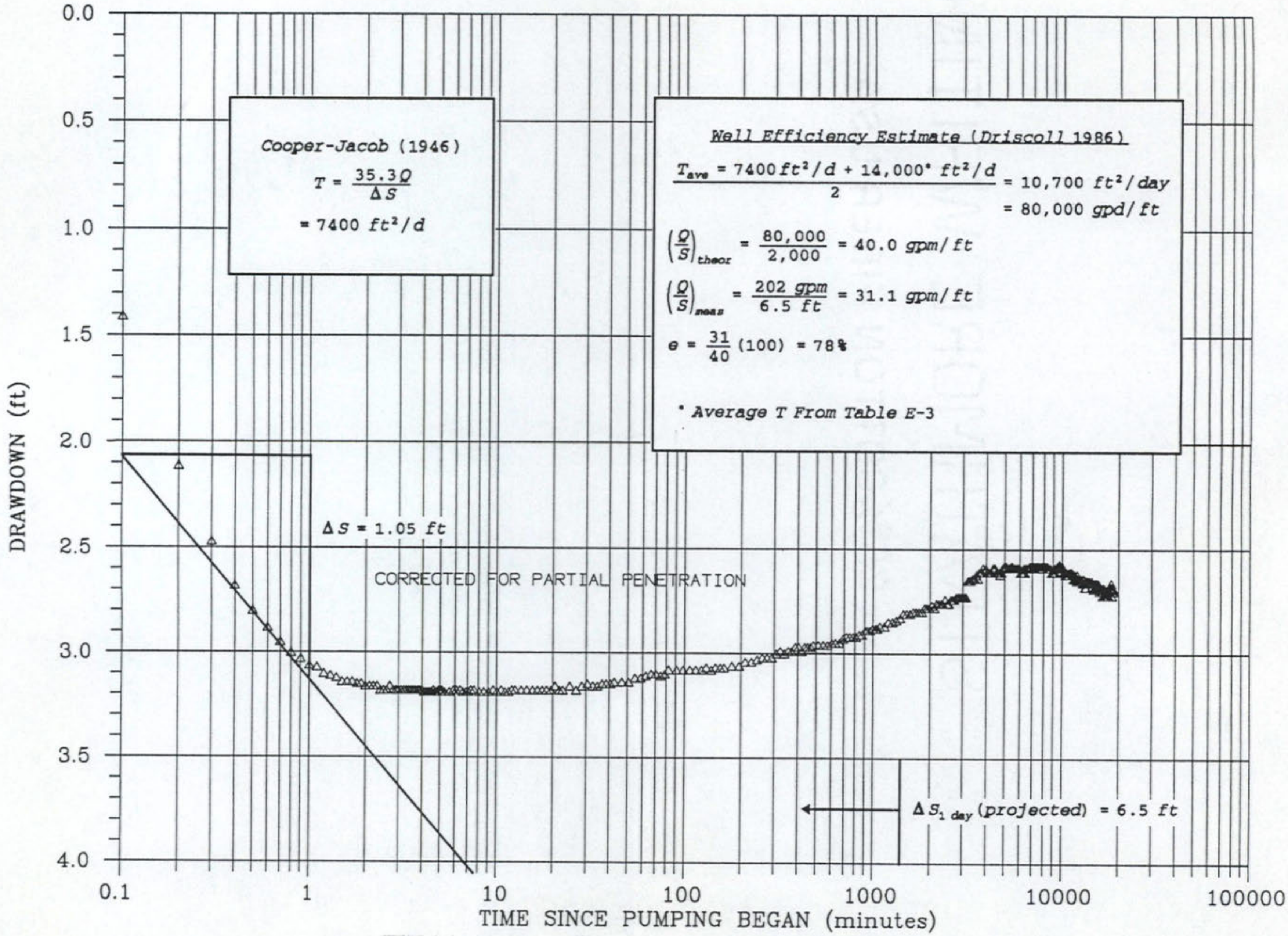

Figure E-40

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